Section 8: Commentary
SEISMIC DESIGN REQUIREMENTS (SDR) 4, 5 AND 6

C8.2 DESIGN FORCES

C8.2.1 Ductile Substructures (R>1) — Flexural Capacity

The key element in the design procedure is the flexural capacity of the columns. Philosophically the lower the flexural capacity of the column the more economic the seismic design provisions because the overstrength flexural capacity of a column drives the cost and capacity of the foundations and the connection to the superstructure. For SDAP B the capacity of the column designed for non-seismic loads is considered to be acceptable for the lower seismic hazard levels.

For SDAP C the design procedure provides a trade-off between acceptable design displacements and minimum flexural capacities of columns. For SDAP D and E the flexural capacity of a column must meet the maximum of the moments from either the 50% PE in 75 years event or the 3% PE in 75 year/1.5 mean deterministic event divided by the appropriate R-Factor. For SDAP C, D, and E there are additional strength limitations based on P-Δ considerations.

C8.2.2 Capacity Protected Elements

The objective of these provisions for conventional design is that inelastic deformation (plastic hinging) occurs at the location in the columns (top and/or bottom) where they can be readily inspected and/or repaired. To achieve this objective all members connected to the columns, the shear capacity of the column and all members in the load path from the superstructure to the foundation, shall be capable of transmitting the maximum (overstrength) force effects developed by plastic hinges in the columns. The exceptions to the need for capacity design of connecting elements is when all substructure elements are designed elastically (Article 4.9), seismic isolation design (Article 8.10) and in the transverse direction of columns when a ductile diaphragm is used (Article 8.7.8.2).

C8.2.3 Elastically Designed Elements

If all the supporting substructures elements (columns, piers, pile bents) are designed elastically, there will be no redistribution of lateral loads due to plastic hinges developing in one or more columns. As a consequence the elastic analysis results are appropriate for design. The recommended provisions attempt to prevent any brittle modes of failure from occurring.

If only one or a selected number of supporting substructure elements are designed elastically, there will be a significant redistribution of lateral loads when one or more of the columns develop plastic hinges. Generally, the elastically designed elements will attract more lateral load. Hence the need to either use capacity design principles for all elements connected to the elastically designed column. If this is not practical, the complete bridge needs to be reanalyzed using the secant stiffness of any columns in which plastic hinges will form in order to capture the redistribution of lateral loads that will occur.

C.8.2.4 Abutments and Connections

In general the connections between the superstructure and substructure should be designed for the maximum forces that could be developed. In the spirit of capacity design, this implies that the forces corresponding to the full plastic mechanism (with yielding elements at their overstrength condition) should be used to design the connections. In cases where the full plastic mechanism might not develop during the 3% in 75-year earthquake, the elastic forces of this event are permitted. However, it is still good practice to design the connections to resist the higher forces corresponding to the full plastic mechanism. It is also good practice to design for the best estimate of forces that might develop in cases such as pile bents with battered piles. In such bents the connections should be stronger than the expected forces, and these forces may be quite large and may have large axial components. In such cases, the plastic mechanism may be governed by the pile geotechnical strengths, rather than the piles' structural strengths.

C8.2.5 Single Span Bridges

Requirements for single span bridges are not as rigorous as for multi-span bridges because of their favorable response to seismic loads in past earthquakes. As a result, single span bridges need not be analyzed for seismic loads regardless of the SDR and design requirements are limited to minimum seat widths and connection forces. Adequate seat widths must be provided in both the transverse and longitudinal directions. Connection forces based on the premise that the bridge is very stiff and that the fundamental period of response will be short. This assumption acknowledges the fact that the period of vibration is difficult to calculate because of significant interaction with the abutments.

These reduced requirements are also based on the assumption that there are no vulnerable substructures (i.e., no columns) and that a rigid (or near rigid) superstructure is in place to distribute the in-plane loads to the abutments. If, however, the superstructure is not able to act as a stiff diaphragm and sustains significant in-plane deformation during horizontal loading, it should be analyzed for these loads and designed accordingly. Single span trusses may be sensitive to in-plane loads and the designer may need to take additional precautions to ensure the safety of truss superstructures.

C8.3 DESIGN DISPLACEMENTS

C8.3.2 Minimum Seat Width Requirement

Unseating of girders at abutments and piers must be avoided in all circumstances. The current Division I-A requirement for minimum seat width is:

$$N = 0.20 + 0.0017L + 0.0067H$$

for seismic performance catergories A and B. The seat width is multiplied by 1.5 for SPC C and D. The seat width is further multiplied by 1/cosα to account for skew effects. The current expression gives reasonable minimum seat widths, but it is modified herein for larger seismic zones.

The requirement for minimum seat width accounts for (1) relative displacement due to out-of-phase ground motion of the piers, (2) rotation of pier footings, and (3) longitudinal and transverse deformation of the pier. The current expression provides reasonable estimates of the first two effects, but underestimates the third. The maximum deformation demand is given by the P–Δ limitation because P–Δ generally controls the displacement of the piers. The capacity spectrum gives:

$$C_c\Delta = \left(\frac{F_v S_1}{2\pi B}\right)^2 g$$

and the P–Δ limitation is:

$$C_c > 4\frac{\Delta}{H}$$

Combining the two expressions gives the maximum displacement when P–Δ controls:

$$\Delta = \frac{\sqrt{g}}{4\pi B}\sqrt{H} \cdot F_v S_1$$

Assuming B=1.4, with moderate ductility capacity, the longitudinal displacement limit in meter units is $\Delta_S = 0.18\sqrt{H} \cdot F_v S_1$.

Transverse displacement of a pier supporting a span with fixed bearing and a span with a longitidinal release will result in additional seat displacement. The seat displacement at the edge of the span with the longitudinal release is $2\Delta_S B / L$. Combining the seat displacement due to longitudinal and transverse displacement of the pier using the SRSS combination rule gives the pier displacement contribution to seat width as:

$$N = 0.18\sqrt{H}\sqrt{1 + \left(2\frac{B}{L}\right)^2} \cdot F_v S_1$$

For $F_v S_1 = 0.40$ the coefficent is 0.072. Because transverse displacement of a pier is limited by "arching" of the superstructure, the maximum of B/L=3/8 is reasonable for determing the seat displacement.

Using this approach, the minimum seat width in Equation 8.3.2-1 is a linear function of the seismic hazard, $F_v S_1$. The factor on seat width varies from unity for $F_v S_1 = 0$ to 1.5 for $F_v S_1 = 0.40$. The factor for $F_v S_1 = 0.80$ is 2.0. The coefficient for the pier deformation term provides a contribution to the seat width for $F_v S_1 = 0.40$ of:

$$N = 0.075\sqrt{H}\sqrt{1 + \left(2\frac{B}{L}\right)^2}$$

which is close the to value from the the P-Δ analysis. The constant term is reduced from 0.20 to 0.10 because the pier deformation is included directly.

Equation (8.3.2-1) provides seat widths that are slightly larger than the Division I-A requirement for low seismic zones and larger seat widths for $F_v S_1 = 0.80$ are larger by a factor of 1.5 to 1.8.

C8.3.3 Displacement Compatibility

Certain components may be designed to carry only dead and live loads (e.g. bearings, non-participating bents, etc.). Other components are non-structural, but their failure would be unacceptable or could result in structural problems (e.g. large diameter water pipes that could erode away soils if they failed). Under seismic loads these components must deform to remain compatible with their connections. The purpose of this section is to require a check that the non-seismic load resisting components have sufficient deformation capacity under seismically induced displacements of the bridge.

C8.3.4 P-Δ Requirements

Structures subject to earthquake ground motion may be susceptible to instability from P-Δ effects. Inadequate strength can result in "ratcheting" of structural displacement, with large residual deformation, and eventually instability. The intent of this section is to provide a minimum strength, or alternatively, a maximum displacement, for which P-Δ effects will not significantly affect seismic behavior of a bridge.

P-Δ produces a negative slope in a structures' force-displacement relationship equal to P/H.

The basis for the requirement in Equation 8.3.4-1 is that the maximum displacement is such that the reduction in resisting force is limited to a 25 percent reduction from the lateral strength assuming no post yield stiffness:

$$\Delta \frac{P}{H} < 0.25V \qquad \text{(C8.3.4-1)}$$

where P is the gravity load on the substructure. Stating a limitation on displacement in terms of lateral strength is justified from dynamic analysis of SDOF systems with various hysteretic relationships. The requirement of Equation (C8.3.4-1) will avoid "ratcheting" in structures with typical post-yield stiffness. The requirement has been shown to limit P-Δ effects from dynamic analysis of single degree-of-freedom systems (Mahin and Boroschek, 1991, MacRae 1994).

The lateral strength can be expressed in terms of the seismic coefficient, $C_c = V/W$, which upon substitution into Equation C8.3.4-1 gives:

$$\Delta \le 0.25C_c\left(\frac{W}{P}\right)H \qquad \text{(C8.3.4-2)}$$

where W is the weight of the bridge responding to horizontal earthquake ground motion. For bridges in which the weight responding to horizontal ground motion is equal to gravity load on the substructure, Equation C8.3.4-2 gives Equation 8.3.4-1.

However, bridges with abutments may have a W/P ratio greater than unity if the abutments do not deform significantly, thus reducing P-Δ effects because a portion of the gravity load is resisted by the abutments. The Engineer may consider using Equation C8.3.4-2 with $W/P \le 2$ when such an assumption is documented.

Equation 8.3.4-1 can also be stated as a minimum seismic coefficient to avoid P-Δ effects.

$$C_c > 4\frac{\Delta}{H} \qquad \text{(C8.3.4-3)}$$

In the short period range, the equal displacement rule does not apply. Inelastic displacement will be greater than the elastic displacement according to:

$$\Delta_{inelastic} = \frac{R_B}{R}\Delta \qquad \text{(C8.3.4-4)}$$

in which R_B is the target reduction factor and R is the ratio of the lateral strength to the elastic force according to Article 4.7. Substitution of Equation 4.7-1 into C8.3.4-4 gives Equations 8.3.4-2 and 8.3.4-3.

C8.3.5 Minimum Displacement Requirements for Piers and Bents

The requirement in this section is based on the "equal displacement rule", that is the maximum displacement from dynamic analysis with a linear model using cracked section properties is approximately equal to the maximum displacement for the yielding structure – Figure C3.3-2.

The factor of 1.5 on the displacement demand recognizes the approximations in the modeling for the seismic analysis. Furthermore, the demand analysis is performed for a model of the entire bridge including three-dimensional effects. However, the displacement capacity verification is done using a two-dimensional pushover analysis on individual bents. Since the relationship between the two methods of analysis is not well-established, the factor of 1.5 represents a degree of conservatism to account for the lack of a rigorous basis for comparing displacement demand and capacity.

For very regular bridges satisfying the requirements for SDAP C in Article 4.4, the displacement requirement implied in the capacity spectrum approach does not include the 1.5 factor.

C8.4 FOUNDATION DESIGN REQUIREMENTS

C8.4.1 Foundation Investigation

C8.4.1.1 General

The conduct of the subsurface exploration program is part of the process of obtaining information relevant for the design and construction of substructure elements. Information from the subsurface exploration is particularly critical in areas of higher seismicity as information from the exploration will determine the Site Classification for seismic design and the potential for geologic hazards, such as liquefaction and slope stability.

The elements of the process that should precede the actual exploration program include search and review of published and unpublished information at and near the site, a visual site inspection, and design of the subsurface exploration program. Refer to AASHTO *Manual on Subsurface Investigations* (1988) for general guidance regarding the planning and conduct of subsurface exploration programs.

C8.4.1.2 Subsurface Investigations

The exploration phase of the project should be conducted early enough that geologic conditions that could have a significant effect on project costs are identified. If subsurface information is not available from previous work in the area, it may be desirable to conduct a limited exploration program before TS&L to identify conditions that may change either the location or type of bridge.

A variety of subsurface exploration methods are available. The most common methods involve drilling methods or cone penetrometer soundings. In some cases geophysical methods can be used to provide information relevant to the design of the substructure system. Appendix B provides a discussion of these methods. As noted in this Appendix, each of these methods has limitations. A geotechnical engineer or engineering geologists should be involved in the selection of the most appropriate exploration method.

8.4.1.3 Laboratory Testing

The equipment and methods used during laboratory testing will depend on the type of soil or rock, as well as the state of disturbance of the sample to be tested. Therefore, the need for certain types of samples should be considered when planning the field exploration phase of the project.

The number and type of laboratory test should be determined after reviewing boring logs developed from the field exploration plan relative to the range in substructures that will be possibly used for the bridge. Additional details regarding laboratory testing are presented in Appendix B.

C8.4.2 Spread Footings

During a seismic event, the inertial response of the bridge deck results in a transient horizontal force at the abutments and central piers. This inertial force is resisted by (1) the abutments, (2) the interior piers, or (3) some combination of the two. Forces imposed on the interior columns or piers result in both horizontal shear force and an overturning moment being imposed on the footing. The footing responds to this load by combined horizontal sliding and rotation. The amount of sliding and rotation depends on the magnitude of imposed load, the size of the footing, and the characteristics of the soil.

For seismic design of spread footings, the response of the footing to shear forces and moment is normally treated independently; i.e., the problem is de-coupled. The overturning component of the column load results in an increase in pressures on the soil. Since the response to moment occurs as a rotation, pressure is highest at the most distant point of the footing, referred to as the toe. This pressure can temporarily exceed the ultimate bearing capacity of the soil. As the overturning moment continues to increase, soil yields at the toe and the heel of the footing can separate from the soil, which is referred to as liftoff of the footing. This liftoff is temporary. As the inertial forces from the earthquake change direction, pressures at the opposite toe increase and, if moments are large enough, liftoff occurs at the opposite side. Bearing failure occurs when the force induced by the moment exceeds the total reactive force that the soil can develop within the area of footing contact. Soil is inherently ductile, and therefore, yielding at the toe and liftoff at the heel of the footing are acceptable phenomena, as long as (1) global stability is preserved and (2) settlements induced by the cyclic loading are small.

The shear component of column load is resisted by two mechanisms: (1) the interface friction between the soil and the footing along the side and at the base of the footing, and (2) the passive resistance at the face of the footing. These resistances are mobilized at different deformations. Generally, it takes more displacement to mobilize the passive pressure. However, once mobilized, it normally provides the primary resistance to horizontal loading.

Various approaches are available to evaluate the response of the bridge-footing system during the design event. In most cases the bridge designer will use equivalent linear springs to represent the soil-footing system. Guidance provided in these Specifications focuses on these simple procedures.

For critical or irregular bridges more rigorous modeling is sometimes used. These methods can involve use of two- and three-dimensional finite element or finite difference modeling methods. This approach to modeling involves considerable expertise in developing a model that represents the soil-structure system. Close cooperation is required between the structural engineer and the geotechnical engineer when developing the model; each discipline has to be familiar with the limitations associated with the development of the model. Without this cooperative approach, it is very easy to obtain very precise results that have little relevance to likely performance during the design earthquake.

Liquefaction represents a special design problem for spread footings because of the potential for loss in bearing support, lateral movement of the soil from flow or lateral spreading, and settlements following an earthquake as porewater pressures in liquefied soil dissipate. Nonlinear, effective stress methods are normally required to adequately replicate these conditions in computer models. Such modeling methods are limited in number and require significant expertise. They are usually applicable for bridge design projects only in special circumstances.

C8.4.2.1 Spring Constants — for Footings

A Winkler spring model is normally used to represent the vertical and moment-rotation curve in the analysis. A uniformly distributed rotational stiffness can be calculated by dividing the total rotational stiffness of the footing by the moment of inertia of the footing in the direction of loading. Similar methods are used for vertical stiffness.

<u>Strain and Liftoff Adjustment Factors</u>

Equations given in Tables 8.4.2.1-1 and 8.4.2.1-2 are based on elastic halfspace theory. These equations were originally developed for very low levels of dynamic loading associated with machine foundations. For these levels of loading, it is possible to use the low-strain shear modulus (G_{max}) of the soil, and the footing remains in full contact with the soil. During seismic loading, at least two different phenomena occur which are inconsistent with the assumptions used in the original development of these equations. These differences involve (1) the nonlinear response of the soil from both free field earthquake wave propagation and from local strain amplitude effects and (2) the liftoff of the footing.

- Strain Amplitude Effects: The strain amplitude effects reflect the inherent nonlinearity of soil, even at very low shearing strain amplitudes. As the seismic wave propagates through the soil, the soil softens, resulting in a reduced shear modulus. Both field measurements and numerical modeling have shown this softening, as discussed by Kramer (1996). A second source of soil nonlinearity also must be considered. As the footing responds to inertial loading from the bridge column, local soil nonlinearities occur around the footing as the soil is subjected to stress from the shear forces and overturning moments. While various procedures exist for estimating the free-field effects of wave propagation, simple methods for estimating the local strain effects have yet to be developed. Nonlinear finite element or finite difference methods can be used to evaluate these effects; however, for most bridge studies such modeling cannot be justified. In recognition of the need for simple guidelines, G/Gmax adjustment factors were estimated. This approach for dealing with soil nonlinearity involves considerable judgment, which may warrant modification on a case-by-case basis.
- Liftoff Effects: The consequence of uplift during seismic loading will be that the effective area of the footing will be less than if full contact were to occur. The amount of uplift is expected to be larger in a higher seismic zone and during an event with a longer return period. The area adjustments for liftoff were made by recognizing that the maximum liftoff allowed under the extreme loading condition will usually be one-half uplift of the footing. It was also recognized that the maximum uplift would only occur for a short period of time, and that during most of the earthquake, the maximum loading might be from 50 to 70 percent of the peak value. For this reason the effective uplift would not be as much as the peak uplift. Values shown were selected after discussing the potential values of effective area that might occur and then applying considerable engineering judgment.

Uncertainty in Spring Constant Determination

Stiffness constants developed in the manner described in this Article involve uncertainty. A prudent Designer will account for this uncertainty by evaluating stiffness for upper and lower bound modulus values, in addition to the best-estimate shear modulus. The upper and lower bound values are used to account for (1) the variability of shear modulus that is likely to occur in the field, (2) the uncertainty in adjustments being used for shearing strain and geometric nonlinearities, and (3) limitations in the equation for determining stiffness.

The range of modulus variation used by the Designer in a sensitivity evaluation is expected to change, depending on the characteristics of the site, the details of the site characterization process, and the type of analysis. Common practice is often to assume that the lower bound shear modulus is approximately 50 percent of the best estimate and the upper bound is approximately 100 percent greater than the best estimate. If the resulting upper and lower bound values of stiffness are such that significant differences in bridge response are possible, then consideration should be given to either (1) evaluating bridge response for the range of stiffness values or (2) performing additional site characterization studies to reduce the range used in defining the upper and lower bound.

Geometric or Radiation Damping

The conventional approach during the use of elastic halfspace methods accounts for energy loss within the foundation system through a spectral damping factor. The spectral damping factor is typically defined as 5 percent, and is intended to represent the damping of the structure-foundation system. This damping differs from the geometric or radiation damping of a foundation. For translational modes of loading, the foundation damping can be in excess of 20 percent. The 5 percent spectral damping used in the modal analysis procedures is intended to account for the geometric damping within the foundation system, as well as damping in the bridge structure. While it may be possible to increase the spectral damping of the overall system to a higher level to account for the high geometric damping within the foundation, in view of the liftoff that is allowed to occur during the design earthquake, it is generally not prudent to count on the high levels of foundation damping, at least without special studies that properly account for the liftoff of the foundation.

C8.4.2.2 Moment-Rotation and Shear-Displacement Relationships for Footings

The foundation capacity requires an evaluation of the soil to resist the overturning moment and the shear force from the column. Vertical loading to the footing will also changed during seismic loading, and this change also needs to be considered.

The initial slope of the moment-rotation curve should be established using the best-estimate rotational spring constant defined in the previous article. Checks can be performed for the upper and lower bound of the initial slope; however, these variations will not normally be important to design.

It is critical during determination of the moment capacity for the moment-rotation curve to use the ultimate bearing capacity for the footing without use of a resistance factor (i.e., use ϕ = 1.0). The determination of ultimate bearing capacity should not be limited by settlement of the footing, as is often done for static bearing capacity determination. The ultimate capacity for the moment-rotation relationship should be defined for the best-estimate soil conditions.

For important bridges, the Designer should consider use of upper and lower bounds for bearing capacity to account for uncertainties. The range for the upper and lower bound will depend on the variability of soils at the site and the extent of field explorations and laboratory testing. Common practice is often to assume that the lower bound capacity is approximately 50 percent of the best estimate and the upper bound is approximately 100 percent greater than the best estimate.

<u>Shear-Displacement</u>

During horizontal shear loading, the resisting force comprises the resistance developed along the base and the sides of the footing and from the passive pressure at the face of the footing. The passive pressure will often provide most of the reaction during a seismic event. For simplicity it can be assumed that the maximum resistance (passive + base + two sides) is developed at a deformation equal to 2 percent of the footing thickness.

The shear resistance on the base and side of the footing should be determined using an interface shear strength. For most cast-in-place concrete foundations, a value of interface friction of 0.8 times the friction angle of the soil will be appropriate. Displacements to mobilize this resistance will normally be less than 10 to 20 mm. The passive pressure at the face of the footing should be computed assuming an interface friction angle equal to 50 percent of the friction angle of the backfill material. The log spiral or Caquot-Kerisel (1948) methods should be used for determining the ultimate passive pressure. If the backfill material changes within twice the height of the footing, the effects of the second material should be included in the computation of the passive pressure. A method of slices similar to a slope stability analysis offers one method of accomplishing this computation.

Deformations needed to mobilize the ultimate passive resistance of the face of a footing could easily exceed 25 mm for a typical footing thickness. The potential consequences of this movement relative to column behavior will usually be evaluated during the soil-structure interaction analysis. The uncertainty in computing deformations associated with ultimate passive resistance determination is such that a variation of –50 percent and +100 percent would not be unusual. If this variation has a significant effect on, say, the push-over-analysis, the Designer may want or modify the foundation or the soil conditions to reduce the uncertainty or limit the deformations.

As discussed by Kramer (1996), evidence exists that the available ultimate passive resistance during seismic loading could be reduced by the seismic response of the ground. This condition occurs if the direction of loading from the inertial response of the bridge structure is the same as the motions in the ground. These two loadings normally occur at dissimilar frequencies, and therefore, the coincidence of the directions of loading is usually for only a moment in time. When the movements are out of phase, the loading increases. It was felt that reducing the passive ultimate resistance for the short periods of coincidence would underestimate the effective passive capacity of the foundation (i.e., low ultimate resistance), and therefore the approach taken in this Specification is to ignore this potential effect. This approach clearly involves considerable judgment, and therefore, an alternate approach that includes the reduction in passive resistance could be used, subject to the Owner's approval.

Vertical Load Capacity

For most designs it is unnecessary to consider increases in vertical forces on the footing during seismic loading, as these forces will normally be a fraction of the gravity load. However, if the bridge site is located in proximity to an active fault, vertical accelerations could become important, as discussed in Article 3.4.5. For these situations the potential displacement should be checked using the spring constants given in Table 8.4.2.1-1 together with the increase in vertical column load. The potential consequences of reduction in vertical loads through inertial response should also be considered. This effect could temporarily decrease lateral resistance and moment capacity

Liquefaction below a spread footing foundation located in SDR 4 and above could be significant because of the combination of higher ground accelerations and larger earthquake magnitudes. As the potential for liquefaction increases, the potential for damage or failure of a bridge from loss in bearing support, lateral flow or lateral spreading of the soil, or settlements of the soil as porewater pressures in the liquefied layers dissipate also increases.

Additional discussion of the consequences of liquefaction are provided in Article 8.6 and Appendix D to these Specifications. A flow chart showing the methodology for addressing the moving soil case is given in Figure D.4.2-1.

C8.4.3 Driven Piles

C8.4.3.1 General

If batter piles are used, consideration must be given to (1) downdrag forces caused by dissipation of porewater pressures following liquefaction, (2) the potential for lateral displacement of the soil from liquefaction-induced flow or lateral spreading, (3) the ductility at the connection of the pile to the pile cap, and (4) the buckling of the pile under combined horizontal and vertical loading. These studies will have to be more detailed than those described elsewhere within Article 8.4. As such, use of batter piles should be handled on a case-by-case basis. Close interaction between the geotechnical engineer and the structural engineer will be essential when modeling the response of the batter pile for seismic loading.

For drained loading conditions, the vertical effective stress, σ'_v, is related to the groundwater level and thus affects pile capacity. Seismic design loads will have a very low probability of occurrence. This low probability normally justifies not using the highest groundwater level during seismic design.

C8.4.3.2 Design Requirements

During a seismic event, the inertial response of the bridge deck results in a transient horizontal force. This inertial force is resisted by (1) the abutments, (2) the interior piers, or (3) some combination of the two. Forces imposed on the interior columns or piers result in both horizontal shear force and overturning moments being imposed on the pile foundation. The pile foundation responds to this load by combined horizontal deflection and rotation. The amount of horizontal deflection and rotation depends on the magnitude of imposed load, the size and type of the foundation system, and the characteristics of the soil.

For seismic design of driven pile foundations, the response of the foundation system to shear forces and moment is normally treated independently; i.e., the problem is de-coupled. If the driven pile is part of a group of piles, as normally occurs, the overturning component of the column load results in an increase in vertical loading on the piles in the direction of loading and a reduction in load in the other direction. Since the response to moment occurs as a rotation, load increase is highest at the most distant pile. This load can temporarily exceed the bearing capacity of the soil. As the overturning moment continues to increase, soil yields at the leading edge of the pile group and the pile begins to plunge. At the trailing edge, uplift loads occur, possibly, resulting in separation between the pile tip and the soil. This uplift is temporary. As the inertial forces from the earthquake change direction, loads at the opposite side increase and, if moments are large enough, uplift occurs at the opposite end. Plunging failure of the pile group occurs only when the force induced by the moment exceeds the total reactive force that the soil can develop for the entire group of piles. Soil is inherently ductile, and therefore, yielding of the forward pile and uplift at the trailing pile are acceptable phenomena, as long as global stability is preserved.

The shear component of column load is resisted by the passive pressure at the face of each pile. Normally, this resistance is mobilized in the upper 5 to 10 pile diameters. If the foundation system includes a pile cap, the reaction to the shear load results from the resistance of the piles and the resistance of the pile cap. The cap develops resistance from (1) the interface friction between the soil and the cap along the side of the cap and (2) the passive resistance at the face of the cap. These resistances are mobilized at different deformations. Generally, it takes more displacement to mobilize the passive pressure. However, once mobilized, it can provide the primary resistance of the foundation system.

For some sites the potential occurrence of scour around the pile is possible. If scour occurs the effective length of the pile could change, which could in turn affect the seismic response of the bridge-foundation system. If a potential for scour around the piles exists during the design life of the bridge, the seismic analysis should be made considering the likely, but not necessarily maximum, depth of scour. In this situation, the maximum depth of scour may not be required because of the low probability of both cases occurring at the same time. If the assumptions on scour depth have (or could have) a significant effect on seismic response, the Designer should meet with the Owner and establish a strategy for addressing this issue. This strategy could involve conducting a series of parametric studies to bracket the range of possible responses.

Similar to the discussion in Article C8.4.2, various approaches are available to evaluate the response of the bridge-foundation system during the design event. In most cases the Designer will use equivalent linear springs to represent the soil-foundation system. Guidance provided in these Specifications focuses on these simple procedures. Comments provided in Article C8.4.2 regarding more rigorous modeling methods are equally valid for pile foundation systems.

Most recent research on seismic response of pile-supported foundations has focused on lateral pile loading. Lam et al. (1998) report that many pile-supported foundations are more sensitive to variations in axial pile stiffness, and therefore, the axial pile-load stiffness problem warrants more consideration. Moment demand on a pile group also generally should govern foundation design, which is determined by axial response of the group, rather than lateral loading for most soil conditions.

Characterization of the stiffness of an individual pile or pile group involves an evaluation of the pile load-displacement behavior for both axial and lateral loading conditions. The overall pile-soil stiffness can be estimated in a number of ways, and the method used should reflect the soil characteristics (e.g., type, strength, and nonlinearity) and the structural properties of the pile or pile group (e.g., type, axial and bending stiffness, diameter, length, and structural constraints). If a stiffness matrix it used, it is critical that it be positive-definite and symmetric for it to be suitable for implementation in a global response analyses. This will require p-y curves to be linearized prior to assembly of the stiffness components of the matrix. Such a procedure has been adopted in the charts shown in Article 8.4.3. If the stiffness matrix is used in a computer program to determine foundation loading demands, then programs such as LPILE or GROUP should be used to determine bending moments and shear forces for design, with nonlinear p-y curves used as appropriate.

The seismic displacement capacity verification step described in Article 5.4.3 requires development of moment-rotation and lateral load-displacement relationships. These relationships are normally assumed to be uncoupled because the lateral loads are mobilized in the upper portion of the pile while the axial load is mobilized at relatively deep elevations. For most push-over analyses a secant stiffness can be used to represent soil springs. If design uplift or plunging limits are exceeded, nonlinear springs should be used. In most cases a bi-linear spring will be an acceptable model of the nonlinear behavior of the soil.

C8.4.3.3 Axial and Rocking Stiffness for Driven Pile/Pile Cap Foundations

Axially loaded piles transfer loads through a combination of end bearing and side resistance along the perimeter of the pile. Their true axial stiffness is a complex nonlinear interaction of the structural properties of the pile and the load-displacement behavior of the soil for friction and end bearing (Lam et al., 1998). Both simplified and more rigorous computer methods are used for evaluating axial stiffness. In most cases simplified methods are sufficient for estimating the axial stiffness of piles. However, at sites where the soil profile changes appreciably with depth or where the effects of group action occur, computer models will often provide a better representation of soil-pile interaction.

Use of Simplified Methods

The axial stiffness value in the simplified equation, $K_{sv} = \Sigma 1.25AE/L$, represents an average value that accounts for uncertainties in the determination of soil properties, the mechanism for developing resistance (i.e., side resistance versus end bearing), and the simplified computational method being used. The basis of this equation is summarized as follows:

- If the pile develops reaction from purely end bearing, the tip bearing stiffness must be relatively large compared with the side resistance stiffness of the soil and the axial stiffness properties of the pile. If the tip displacement is assumed to be zero, the resulting axial stiffness is

$$K_{sv} = \Sigma\ AE/L$$

- At the other extreme, a purely friction pile implies that the force at the tip is zero. For zero tip force and a uniform total transfer to the soil by side resistance along the pile, the axial stiffness of the pile approximates:

$$K_{sv} = \Sigma\ 2AE/L$$

- Allowing for some tip displacements and recognizing the inherent complexity of the problem, a reasonable range is from 0.5AE/L to 2AE/L.

Other methods of estimating the axial stiffness of the pile are also available. Lam et al. (1998) present a simplified graphical procedure that uses the average between a rigid and flexible pile solution.

Nonlinear Computer Methods

In the above discussion simplified methods are used to define the axial stiffness of a single pile. More rigorous computer methods that accommodate the nonlinear behavior of the soil and structure are also available. These more rigorous methods involve more effort on the part of the Designer. In many cases the increased accuracy of the more rigorous method is limited by the uncertainty associated with selection of input parameters for the analyses.

A number of computer programs are available for conducting more rigorous determination of the axial stiffness of the soil-pile system (e.g., Lam and Law, 1994). These programs are analogous to the program used to estimate the lateral load-displacement response of piles. Rather than "p-y" curves, they use "t-z" curves and "q-z" curves to represent the side resistance and end bearing load-displacement relationships, respectively.

These same procedures can be used to determine uplift stiffness values. For these determinations the end bearing component of the load-displacement relationship is deleted, and the resistance in uplift is assumed to be the same as that in compression.

Computer programs such as APILE Plus (Reese et al., 1998) provide recommendations for load-transfer relationships in end bearing and side resistance for driven piles. Typical amounts of displacement to mobilize side resistance are on the order of a few millimeters in sands and up to 2 percent of the pile diameter in clay. According to

Reese et al. (1998), up to 10 percent of the pile diameter can be required to mobilize the full end bearing of a pile, whether it is in clay or sand. Actual determination of the deformations to mobilize either end bearing or side resistance involves considerable judgment. While the computer programs often make the material property selection and the analysis procedure easy, the uncertainty of the analysis can still be very large. For this reason it is important to involve a person knowledgeable in soil properties and pile loading in the selection of the soil parameters used to model the load-displacement relationship.

The effects of group action for axial loading can be modeled in some computer programs by modification of “t-z” and “q-z” curves. The modifications to these curves will depend on the soil type, with cohesionless soils showing increasing stiffness as the spacing decreases and cohesive soils softening with decreasing spacing. In contrast to lateral loading, explicit relationships for modifying the “t-z” and “q-z” curves are not provided. However, in the limit the adjusted curves should result in an ultimate capcity similar to ultimate capacity of a group determined by static methods (i.e., $Q_g = nQ_s*\eta$ where Q_g is the capacity of the group, n is the number of piles in the group, Q_s is the capacity of an isolated pile, and η is an efficiency factor that will vary with pile spacing and soil type. In the user’s manual for GROUP (Reese and Wang, 1996), the authors indicate that the efficiency of pile groups in sands is greater than 1 and by implication the stiffness of a closely spaced group will be greater. They also show that the efficiency of pile groups in clays is less than 1, with the implication that the stiffness of a closely space group will be lower.

C8.4.3.4 Lateral Stiffness Parameters for Driven Pile/Pile Cap Foundations

As with axial stiffness, a variety of methods are available for determining the lateral stiffness of a pile or group of piles. Generally, these methods involve the use of simplified charts or the use of more rigorous computer models. The simplified methods normally provide a convenient method for initial design of a pile-supported bridge and may be sufficient for final design if earthquake loads are small. Computer models allow the user to explicitly account for variations in soil stiffness along the embedded depth of the pile, and to account for the effects of group action and changes in the flexural stiffness of the pile during loading. For these reasons, the computer models are often used in final design, particularly where significant changes in soil profile occur with depth or where earthquake loads are large.

<u>Use of Simplified Linear Charts</u>

The charts developed by Lam and Martin (1986) and presented as Figures 8.4.3.4-1 through 8.4.3.4-6 require that an "f" value be defined for the soil. Lam et al. (1998) recommend that the "f" value be selected at a depth of approximately 5 pile diameters. The charts assume no pile top embedment, but yield reasonable stiffnesses for shallow embedment of no more than 1.5 m. Lateral pile stiffness increases quickly with depth for most piles, and therefore, if greater embedment occurs, nonlinear computer methods should be used, as the charts will potentially result in a considerable underestimation of stiffness.

These charts are applicable for pile-head deflections between 5 and 50 mm. The charts also assume that the piles are sufficiently long to achieve full fixity.

<u>Use of Computer Methods</u>

Procedures for conducting nonlinear lateral pile analyses are described by Lam and Martin (1986). Lam and Martin's discussion includes procedures for developing "p-y" curves in both sands and clays. Reese et al. (1997), as well as a number of other technical papers, also discuss the development of "p-y" curves.

A number of these methods identify a factor for cyclic loading. Generally, this factor is not applicable to seismic loading conditions. It was developed for problems involving wave loading to offshore structures, where thousands of cycles of load were being applied. For earthquake problems, the non-cyclic "p-y" curves are most applicable.

Group interaction should usually be considered in the evaluation of lateral response of closely spaced piles. Interaction results when the lateral stress developed during loading of one pile interacts with the adjacent pile. Group reduction curves are usually used to represent this interaction. Early studies suggested significant reduction in stiffness for pile spacings of 8 diameters or less. More recent studies indicate that the group effects are not normally as significant as once thought. A reduction factor of 50 percent is recommended by Lam et al. (1998) as being appropriate for most seismic loading situations. According to Lam et al. (1998), this reduction accounts for the effects of gapping, local porewater pressure effects, and the interaction of the stress field from individual piles. Alternatively, p-multiplier methods suggested by Brown et al. (1988) provide a systematic method of introducing group effects for various pile group configurations.

Another consideration in the use of computer programs is whether a cracked or uncracked section modulus should be used in the representation of concrete piles. This modulus will have a significant influence on the resulting load-deformation response calculation, and therefore requires careful consideration by the person performing the analyses. Programs such as FLPIER and LPILE can explicitly account for the transition from uncracked to cracked section modulus during the loading sequence.

C8.4.3.5 Pile Cap Stiffness and Capacity

The response of the pile-supported footing differs in one important respect from a spread footing foundation: resistance at the base of the footing is not included in the response evaluation. The base resistance is neglected to account for likely separation between the base of the foundation and soil, as soil settlement occurs.

As noted in Article C8.4.2.2, the pile cap will have to deform by as much as 2 percent of the pile cap thickness to mobilize the passive pressure of the cap. If this displacement is significantly greater than the design displacement, it may be possible to neglect the contribution of the pile cap without significant effects on the total stiffness calculation. At these low displacements, the stiffness of the pile will govern response.

C8.4.3.6 Moment and Shear Design

The stiffness of the pile in axial loading is limited by the plunging capacity of the pile. Side resistance and end bearing soil springs should be limited by the unfactored axial capacity at large deformations. Similarly, moment capacity checks are normally made with the unfactored axial capacity of the pile. Resistance factors are not applied to enable the Designer to obtain a better understanding of pile performance under seismic loading. By using unfactored capacities, a best-estimate of the displacement for a given force in the bridge structure can be obtained. If factored capacities are used, the deformation could be greater than under best-estimate conditions, resulting in design decisions that may not be appropriate.

It is recognized that uncertainty exists even with the best-estimate capacity. Although it may not be economical to evaluate these uncertainties in all bridges, uncertainty should be considered during evaluations of stiffness and capacity and should be evaluated for more important bridges. To account for uncertainty, upper and lower bound capacities and stiffnesses can be determined, allowing the Designer to assess the potential effects to design if higher or lower capacities occur for the site.

The range for the upper and lower bound evaluation will depend on the characteristics of the site, the type of analysis used to estimate capacity, and whether or not a field load test is conducted (e.g., PDA, static load test with head measurements only, or fully instrumented pile-load test). Common practice is to use an upper bound that is 100 percent greater than the unfactored stiffness and capacity and a lower bound that is 50 percent of the unfactored stiffness and capacity.

The range of uncertainty is normally higher than the uncertainty implied by the resistance factor used for static design for several reasons: (1) there is greater uncertainty in the seismic resistance of the pile in seismic loading than static loading, (2) there is a greater potential for cyclic degradation of resistance properties during seismic loading, and (3) there are rate of loading effects.

The Designer can reduce the range of uncertainty by conducting more detailed site explorations to fully characterize the soil, by performing more rigorous analyses that treat the full load-deformation process, and by conducting pile-load test to quantify the load-displacement response of the pile. Even with a full-scale field load test, some uncertainty exists as discussed in the previous paragraph. For this reason, a range of values to represent upper and lower bound response may be warranted even under the best circumstances.

C8.4.3.7 Liquefaction and Dynamic Settlement Evaluations

The design of a pile foundation for a liquefied soil condition involves careful consideration on the part of the Designer. Two general cases occur: liquefaction with and without lateral flow and spreading.

Liquefaction without Lateral Flow or Spreading

Pile foundations should be designed to extend below the maximum depth of liquefaction by at least 3 pile diameters or to a depth that axial and lateral capacity are not affected by liquefaction of the overlying layer. Porewater pressures in a liquefied zone can result in increases in porewater within layers below the liquefied zone. Porewater pressures increases can also occur in a zone where the factor of safety for liquefaction is greater than 1.0, as discussed in Appendix 3B. These increases in porewater pressures will temporarily reduce the strength of the material from its pre-earthquake (static) strength. The potential for this decrease should be evaluated, and the capacity of the foundation evaluated for the lower strength. Alternatively, the toe of the pile should be founded at a depth where the effects of porewater pressure changes are small. Normally, the static design of the pile will include a resistance factor of 0.6 or less. This reserve capacity allows an increase in porewater pressures by 20 percent without significant downward movement of the pile.

As porewater pressures dissipate following liquefaction, drag loads will develop on the side of the pile. The drag loads occur between the pile cap and the bottom of the liquefied layer. The side friction used to compute drag loads will increase with dissipation in porewater pressure from the residual strength of the liquefied sand to a value approaching the static strength of the sand. The maximum drag occurs when the porewater pressures are close to being dissipated. Simultaneously relative movement between the pile and the soil decrease as the porewater pressure decreases, resulting in the drag load evaluation being a relatively complex soil-pile interaction problem. For simplicity, it can be conservatively assumed that the drag load used in the settlement estimate is determined by the pre-liquefied side resistance along the side of the pile between the bottom of the pile cap and the bottom of the liquefied zone.

Liquefaction with Lateral Flow or Spreading

Lateral flow and spreading have been common occurrences during liquefaction at bridge sites involving an approach fill or at a river or stream crossing. The amount of movement can range from a few millimeters to over a meter. This amount of movement is generally sufficient to develop full passive pressures on pile or pile cap surfaces exposed to the moving soil. If the pile-pile cap system is not strong enough to resist these movements, the pile cap system will displace horizontally under the imposed load.

Procedures for estimating either the forces and displacements of the pile from the moving ground are discussed in Appendix D. If these forces or displacements are large, some type of ground remediation might be used to reduce these displacements. These ground remediation methods can include vibro densification, stone columns, pressure grouting, or in-place soil mixing. Costs of these improvements can range from \$10/m^3 to in excess of \$40/m^3 (in 2000 dollars). Depending on the specific conditions and design requirements for a site, the use of ground improvement could increase construction costs by 10 percent or more. In view of these costs, the Owner needs to be made aware of the potential risks and the costs of remediation methods as soon as these conditions are identified.

Appendix D provides a more detailed discussion of the process to follow when designing for lateral flow or spreading ground.

For SDR 4 and above, the change in lateral stiffness of the pile resulting from liquefaction is also determined. This change in stiffness is usually accomplished by defining the liquefied zone as a cohesive soil layer with the ultimate strength in the "p-y curve" being equal to the residual strength of the liquefied soil. Appendix D identifies procedures for making these adjustments.

C8.4.4 Drilled Shafts

Lam et al. (1998) provide a detailed discussion of the seismic response and design of drilled shaft foundations. Their discussion includes a summary of procedures to determine the stiffness matrix required to represent the shaft foundation in most dynamic analyses.

Drilled shaft foundations will often involve a single shaft, rather than a group of shafts, as in the case of driven piles. In this configuration the relative importance of axial and lateral response change. Without the pile cap, lateral-load displacement of the shaft becomes more critical than the axial-load displacement relationships discussed for driven piles.

Many drilled shaft foundation systems consist of a single shaft supporting a column. Compressive and uplift loads on these shafts during seismic loading will normally be within limits of load factors used for gravity loading. However, checks should be performed to confirm that any changes in axial load don't exceed ultimate capacities in uplift or compression. In contrast to driven piles in a group, no reserve capacity exists for a single shaft; i.e., if ultimate capacity is exceeded, large deformations can occur.

Special design studies can be performed to demonstrate that deformations are within acceptable limits if axial loads approach or exceed the ultimate uplift or compressive capacities if the drilled shaft is part of a group. These studies can be conducted using computer programs, such as APILE Plus (Reese, et al., 1997). Such studies generally will require rigorous soil-structure interaction modeling.

Various studies (Lam et al., 1998) have found that conventional p-y stiffnesses derived for driven piles are too soft for drilled shafts. This softer response is attributed to a combination of (1) higher unit side friction, (2) base shear at the bottom of the shaft, and (3) the rotation of the shaft. The rotation effect is often implicitly included in the interpretation of lateral load tests, as most lateral load tests are conducted in a free-head condition. A scaling factor equal to the ratio of shaft diameter to 600 mm is generally applicable, according to Lam et al. (1998). The scaling factor is applied to either the linear subgrade modulus or the resistance value in the p-y curves. This adjustment is thought to be somewhat dependent on the construction method.

Base shear can also provide significant resistance to lateral loading for large diameter shafts. The amount of resistance developed in shear will be determined by conditions at the based of the shaft during construction. For dry conditions where the native soil is relatively undisturbed, the contributions for base shear can be significant. However, in many cases the base conditions result in low interface strengths. For this reason the amount of base shear to incorporate in a lateral analyses will vary from case-to-case.

Typically it is necessary to embed shafts to between 2 diameters in rock to 3 and 5 shaft diameters in soil to achieve stable conditions. The depth for stable conditions will depend on the stiffness of the rock or soil. Lower stable lengths are acceptable if the embedment length and the strength of the drilled shaft provide sufficient lateral stiffness with adequate allowances for uncertainties in soil stiffness. Generally, it will be necessary to conduct a lateral load analysis using a program such as COM624 or LPILE to demonstrate that lower stable lengths are acceptable.

Section properties of the drilled shaft should be consistent with the deformation caused by the seismic loading. In many cases it is necessary to use the cracked section modulus in the evaluation of lateral load-displacement relationships. In the absence of detailed information regarding reinforcing steel and applied load, an equivalent cracked section can be estimated by reducing the stiffness of the uncracked section by half. In general the cracked section is a function of the reinforcement ratio (i.e., volume of steel reinforcement versus that of concrete), but is often adequate to assume as one-half of the uncracked section.

C8.5 ABUTMENT DESIGN REQUIREMENTS

C8.5.1 General

One of the most frequent observations of damage during past earthquakes has been damage to the abutment wall. This damage has been due to two primary causes: (1) the approach fill has moved outward, carrying the abutment with it, and (2) large reactive forces have been imposed on the abutment as the bridge deck has forced it into the approach fill. This latter cause of damage has often resulted from a design philosophy that assumed that the abutment wall had to survive only active seismic earth pressures, and that gaps between the bridge deck and abutment wall would not close. In many cases the gap was not sufficient to remain open, and very large loads were imposed by the deck. The passive reaction from the soil was as much as 30 times the forces used for active pressure design, resulting in overloading to and damage of the wall.

These seismic provisions have been prepared to specifically acknowledge the potential for this higher load to the abutment wall. If designed properly, the reactive capacity of the approach fill can provide significant benefit to the bridge-foundation system.

C8.5.2 Longitudinal Direction

Common practice is to use the Mononobe-Okabe equations to estimate the magnitude of seismic earth pressures, for both active and passive pressure conditions. Previous editions of the AASHTO specifications have specifically discussed these methods and presented equations for making these estimates. These equations have, however, been found to have significant limitations.

For the case of seismic active earth pressures, the Mononobe-Okabe equations are based on the Coulomb failure wedge assumption and a cohesionless backfill. For high accelerations or for backslopes, the equations lead to excessively high pressures that asymptote to infinity at critical acceleration levels or backslope angles. For the latter conditions, no real solutions to the equations exist implying equilibrium is not possible (Das, 1999). For horizontal backfills for example, for a friction angle for sand of 40 degrees, a wall friction angle of 20 degrees and a peak acceleration of 0.4g, the failure surface angle is 20 degrees to the horizontal. For a peak acceleration of 0.84g, the active pressure becomes infinite, implying a horizontal failure surface.

Clearly, for practical situations, cohensionless soil is unlikely to be present for a great distance behind an abutment wall and encompass the entire failure wedge under seismic conditions. In some cases, free-draining cohesionless soil may only be placed in the static active wedge (say at a 60 degree angle) with the remainder of the soil being cohesive embankment fill (c,ϕ soil) or even rock. Under these circumstances, the maximum earthquake-induced active pressure should be determined using trial wedges (Figure C8.5.2-1), with the strength on the failure planes determined from the strength parameters for the soils through which the failure plane passes. This approach will provide more realistic estimates of active pressure.

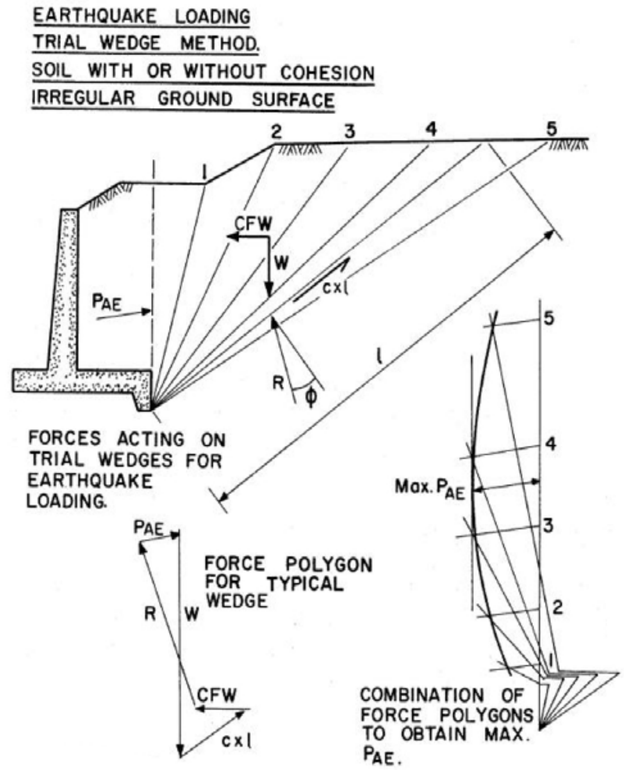


Figure C8.5.2-1 Trial Wedge Method For Determining Critical Earthquake-Induced Active Forces

C8.5.2.1 SDAP B and C

No seismic provisions are required for bridges covered by these SDAPs because increased earth pressures from the approach fill and bridge displacements will normally be within tolerable levels. In the case of seismically induced active earth pressures, the static design of the wall will usually result in the controlling load case, if normal load and resistance factors are used. In the case of integral abutments, the designs based on static at-rest pressures will also be sufficiently conservative to meet seismic demand. For cases where the abutment is engaged and high passive forces could develop, the preferred approach is to design a fuse into the system to protect against damage. Alternatively, the Owner could decide to accept some level of damage, given the low likelihood of occurrence of the design earthquake.

C8.5.2.2 SDAP D and E

The determination of stiffness and capacity is a key step during the design of many bridges by these SDAPs. Procedures for calculating passive force, $P_{p,}$ and abutment stiffness are described below. These procedures should use best-estimate soil properties. The approach is based upon using a uniform distribution of passive soil pressure against the abutment backwall. The uniform pressure approach is a simplification of more complex distribution patterns, which are functions of wall friction and deformation patterns (ie translation or tilting).

C8.5.3 Transverse Direction

To meet the performance criteria, abutments shall experience essentially no damage in the 50% PE in 75-year earthquake, and this may be achieved if the abutments are designed to resist the elastic forces for the 50% PE in 75-year event. For the larger 3% PE in 75-year/1.5 mean deterministic event, the elastic forces may be large enough that they cannot be resisted without some abutment damage. In general, the design of the abutment should attempt to restrict damage to locations that are inspectable and which can be reasonably accessed for repair.

Two preferred strategies may be considered. One is to use isolation, elastomeric or other bearings that accommodate the full seismic movement at the abutment and thereby significantly reduce the likelihood of damage to the abutment itself. The second strategy is to use fuse elements (isolation bearings with a high yield level or shear keys) that are intended to yield or breakaway thereby limiting the forces transferred to the abutment. It should be noted that it is difficult to predict the capacity of a concrete shear key and hence this is a less reliable concept when compared to isolation elements with a high yield force. Such fuse elements should be designed to restrict damage to inspectable locations. In situations where neither of these strategies is practical, then damage may be incurred in the foundation of the abutment, but such a design approach shall only be undertaken with the approval of the Owner.

The calculation of stiffness may require the estimation of effective secant stiffnesses based on ultimate strength and estimates of yield displacements. The approach will be similar to that used in calculating longitudinal abutment stiffness. Alternately, bounding analyses may be used wherein a resisting element is completely released. Where a complete loss of resistance may occur, for example breakaway shear keys or blocks, a small nominal spring resistance may be necessary to obtain reasonable and stable results from a multimode dynamic analysis.

C8.5.3.1 SDAP B and C

For abutments of bridges in the lower seismic design categories, the abutment as typically designed for service loads should be adequate for resisting the seismic effects. Where lateral restraint is provided at the abutment, with for example shear keys, minimum design forces are specified to provide a reasonable amount of strength to resist the forces that are likely to develop in an earthquake.

Abutments designed for non-seismic loads and for the connection forces outlined in Article 4.2 for SDAP A1 and A2 or in Article 4.3 for SDAP B should resist earthquakes with minimal damage. Bridges designed using SDAP C are proportioned such that the abutments are not required to resist inertial forces. Therefore some damage may occur in abutments of such bridges, particularly in the higher Seismic Hazard Levels.

C8.5.3.2 SDAP D and E

For SDAP D, and E, seismic design and analysis is required and the actual restraint conditions at the abutments will determine the amount of force that is attracted to the abutments. These forces shall either be resisted elastically or fuse elements may be used.

Short bridges that have abutments, which can continuously provide soil resistance under cyclic deformations, will exhibit damping that likely exceeds the normal 5 percent value. Therefore for shorter bridges that have small skew and horizontal curves, a 1.4 reduction value is allowed for all the elastic forces and displacements resulting from a transverse earthquake. This provision only applies to shorter bridges with a continuous superstructure where the effects of the transverse abutment response extend throughout the entire bridge. To rely on this reduction, the soil must be able to continuously provide resistance under cyclic loading. Friction against the base of foundations not supported on piles or shafts may be considered sustained resistance, as may be friction against vertical surfaces not subject to gapping as described below. The force reduction is not permitted for other types of abutment resistance, for instance, passive mobilization of backfill where a gap may form between the soil and the backwall. These provisions have been adapted from the "short bridge" provisions outlined by Caltrans in their Seismic Design Criteria and Memo 20-4.

Wingwalls, in general, should not be relied upon to resist significant transverse forces. Typical configurations of wingwalls are normally inadequate to resist large forces corresponding to the passive resistance of the soil retained by the wingwalls. The wingwalls' yield resistance may, however, be counted in the resistance, even though this value will likely not contribute significantly to the lateral resistance.

In cases where the backfill may be displaced passively, whether intended to be part of the ERS or not, the possibility of a gap opening in the backfill should be considered when calculating the transverse lateral capacity of an abutment. If a gap could open between the backfill soil and the abutment, the transverse resistance provided by the wingwalls may be compromised. Specifically, cohesion in the backfill may produce such a situation. If this occurs, reduction of the transverse resistance may be necessary.

C8.6 LIQUEFACTION DESIGN REQUIREMENTS

C8.6.1 General

Liquefaction has been perhaps the single most significant cause of damage to bridge structures during past earthquakes. Most of the damage has been related to lateral movement of soil at the bridge abutments. However, cases involving the loss in lateral and vertical bearing support of foundations for central piers of a bridge have also occurred.

The potential for liquefaction requires careful attention to the determination of the potential for and consequences of liquefaction. For magnitudes less than 6.0, liquefaction develops slowly at most sites, and results in minimal effects to the structure during dynamic shaking, and therefore the effects of liquefaction on dynamic response can be neglected. In addition little potential exists for permanent movement of the ground, again because of the small size and limited duration of seismic events in these areas. If the mean magnitude of the 3% PE in 75 year event is less than 6.0, then the discussion above with regard to duration is applicable. For the magnitude interval of 6.0 to 6.4, a liquefaction analysis is not required when the combination of ground shaking and blow count are below values that would cause liquefaction. This transition interval is based on an assessment of available data from past earthquakes and engineering judgment.

The mean magnitudes shown in Figures 8.6.1-1 to 8.6.1-4 are based on deaggregation information, which can be found in the USGS website (http://geohazards.cr.usgs.gov/eq/). A site-specific determination of the mean magnitude can be obtained from this website using the coordinates of the project site.

If liquefaction occurs in the 50% PE in 75 year event then the performance criteria for piles will need to be operational for the life safety performance level as per Article 8.8.6.2.

C8.6.2 Evaluation of Liquefaction Potential

A site is considered potentially susceptible to liquefaction if one or more of the following conditions exists (SCEC, 1999):

- Liquefaction has occurred at the site during historical earthquakes.
- The site consists of uncompacted or poorly compacted fills containing liquefaction-susceptible materials that are saturated, nearly saturated, or may be expected to become saturated.
- The site has sufficient existing geotechnical data, and analyses indicate that the soils are potentially susceptible to liquefaction.

For sites where geotechnical data are lacking or insufficient, the potential for liquefaction can be delineated using one or more of the following criteria:

- The site consists of soil of late Holocene age (less than 1,000 years old, current river channels and their historical flood plains, marshes, and estuaries) where the groundwater is less than 12 m deep and the anticipated earthquake ground shaking F_aS_s is greater than 0.375 (peak ground acceleration (PGA) greater than 0.15g.)
- The site consists of soils of Holocene age (less than 11,000 years old) where the ground water is less than 10 m below the surface and F_aS_s is greater than 0.50 (PGA is greater than 0.2g.)
- The site consists of soils of latest Pleistocene age (11,000 to 15,000 years before present) where the ground water is less than 5 m below the surface and F_aS_s is greater than 0.75 (PGA is greater than 0.3g).

C8.6.3 Evaluation of the Effects of Liquefaction and Lateral Ground Movement

The design of bridge structures for liquefaction effects generally has two components.

- Vibration Effects: The first is that the bridge must perform adequately with just the liquefaction-induced soil changes alone. This means that the mechanical properties of the soil that liquefy are changed to reflect their liquefied conditions (i.e., "p-y" curves or modulus of subgrade reaction for lateral stiffness are reduced). Design for these cases is in reality a design for structural vibration effects, and these are the effects that the code-based procedures typically cover for design.
- Permanent Displacement Effects: The second component of the design is the consideration of liquefaction-induced ground movements. These can take several forms: lateral spreading, lateral flow, and dynamic settlement. Lateral spreading is a lateral movement that is induced by the ground shaking and develops in an incremental fashion as shaking occurs. Flow, on the other hand, is movement that occurs due to the combined effects of sustained pore pressure and gravity without the inertial loading from the earthquake. Flows can occur several minutes following an earthquake when porewater pressures redistribute to form a critical combination with gravity loading. Dynamic settlement occurs following an earthquake as porewater pressures dissipate.

Vibration and permanent movement occur simultaneously during a seismic event. Their simultaneous occurrence is a complicated process that is difficult to represent without the use of very complex computer modeling. For most bridges the complexity of the modeling doesn't warrant performing a combined analysis. In these cases the recommended methodology is to consider the two effects independently, i.e., de-coupled. The reasoning behind this is that it is not likely that the peak vibrational response and the peak spreading or flow effect will occur simultaneously. For many earthquakes the peak vibration response occurs somewhat in advance of maximum ground movement loading. For very large earthquakes where liquefaction may occur before peak ground accelerations occur, the peak vibration response is like to be significantly attenuated and, hence, inertial loading reduced from peak design values. In addition peak displacements demands arising from lateral ground spreading are likely to generate maximum pile moments at depths well below peak moments arising from inertial loading. Finally, the de-coupling of response allows the flexibility to use separate and different performance criteria for design to accommodate the two phenomena. Two detailed case studies on the application of the recommended design methods for both liquefaction and lateral flow design are given in an NCHRP Report (ATC/MCEER, 2000)

While the de-coupled method is recommended for most bridges, more rigorous approaches are sometimes necessary, such as when a critical bridge might be involved. Coupled approaches are available to represent the large-strain, pore-water pressure buildup mechanisms that occurs during liquefaction. However, these methods are difficult to use, and should only be considered after detailed discussions between the Owner and the Engineer regarding the capabilities and limitations of these methods.

If lateral flow occurs, significant movement of the abutment and foundation systems can result.

Inelastic deformation of the piles is permitted for this condition (e.g., plastic rotation of 0.05 radians). The geometric constraints of Table C3.2-1 provide guidance for meeting the desired performance objective. The range of design options include designing the piles for the flow forces to an acceptance of the predicted lateral flow movements realizing the bridge may need to replaced. Structural and/or soil mitigation measures may be used to minimize the amount of movement to meet higher performance objectives.

C8.6.4 Design Requirements if Liquefaction and Ground Movement Occurs

Spread footings are not normally used in if liquefiable soils are present. Spread footings can be considered if the spread footing is located below the bottom of the liquefiable layer, the ground will be improved to eliminate the potential for liquefaction, or special studies are conducted to demonstrate that the spread footing will perform adequately during and following liquefaction. In most situations these requirements will result in the use of either driven pile or drilled shaft foundations.

The approach used to design the foundation first involves designing to accommodate the non-seismic load conditions and the vibration case of seismic loading without liquefaction. This structure and foundation system should then be assessed for its capability to resist the inertial loads when the soil layers have liquefied. In general this second case will only impact the design of the structure above the foundation system when the upper layers of soil have liquefied.

As noted in Article C7.6.4, lateral flow is one of the more difficult issues to address because of the uncertainty in the movements that may occur. The design steps to address lateral flow are given in Appendix D. A liberal plastic rotation of the piles is permitted, but this does imply that the piles and possibly other parts of the bridge will need to be replaced if these levels of deformation do occur. One suggestion is to use a tube in the center of a few piles so that the amount of subsurface deformation could be measured after an earthquake. Design options range from an acceptance of the movements with significant damage to the piles and columns if the movements are large to designing the piles to resist the forces generated by lateral spreading. Between these options are a range of mitigation measures to limit the amount of movement to tolerable levels for the desired performance objective. Pile group effects are not significant for liquefied soil.

Because the foundation will typically possess some lateral resistance capable of reducing the magnitude of spreading, this capacity should be utilized. If the lateral displacements are too great for the structure to adequately accommodate, then geotechnical improvements will be necessary, unless the performance objective under spreading loads is to accept a severely damaged bridge that likely will need to be replaced. Therefore the most cost-effective approach is to account for the beneficial restraint action of the existing (as-designed for non-spreading effects) foundation.

Additionally, if the foundation can provide significant restraint, but not fully adequate restraint, then additional piles may be considered. Depending on the soil profile and the manner in which spreading develops, simple “pinch” piles provided in addition to the foundation may prove effective. The cost trade-off between pinch piles and geotechnical remediation should be assessed to determine the most effective means of achieving appropriate soil restraint.

C8.6.5 Detailed Foundation Design Requirements

See Article C8.4 for the commentary.

C8.7 STRUCTURAL STEEL DESIGN REQUIREMENTS

C8.7.1 General

It is essential to realize that most components of steel bridges are not expected to behave in a cyclic inelastic manner during an earthquake. The provisions of Article 8.7 are only applicable to the limited number of components (such as specially detailed ductile substructures or ductile diaphragms) whose stable hysteretic behavior is relied upon to ensure satisfactory bridge seismic performance. The seismic provisions of Article 8.7 are not applicable to the other steel members expected to remain elastic during seismic response. Note that in most steel bridges, the steel superstructure is expected (or can be designed) to remain elastic.

Until recently, only a few steel bridges had been seriously damaged in earthquakes. One span of the San Francisco-Oakland Bay Bridge collapsed due to loss of support at its bearings during the 1989 Loma Prieta earthquake, and another bridge suffered severe bearing damage (EERI, 1990). The end diaphragms of some steel bridges suffered damage in a subsequent earthquake in Northern California (Roberts, 1992). During the 1994 Northridge earthquake some steel bridges, located very close to the epicenter, sustained damage to either their reinforced concrete abutments, connections between concrete substructures and steel superstructures, steel diaphragms or structural components near the diaphragms (Astaneh-Asl et al, 1994). However, a large number of steel bridges were damaged by the 1995 Hyogoken-Nanbu (Kobe) earthquake. The concentration of steel bridges in the area of severe ground motion was considerably larger than for any previous earthquake and some steel bridges collapsed. Many steel piers, bearings, seismic restrainers and superstructure components suffered significant damage (Bruneau, Wilson and Tremblay, 1996). This experience emphasizes the importance of ductile detailing in the critical elements of steel bridges.

Research on the seismic behavior of steel bridges (e.g. Astaneh-Asl, Shen and Cho, 1993; Dicleli and Bruneau, 1995a, 1995b; Dietrich and Itani, 1999; Itani et al., 1998a; McCallen and Astaneh-Asl, 1996; Seim, Ingham and Rodriguez, 1993; Uang et al., 2000; Uang et al., 2001; Zahrai and Bruneau 1998) and findings from recent seismic evaluation and rehabilitation projects (e.g. Astaneh and Roberts, 1993, 1996; Ballard et al., 1996; Billings et al, 1996; Dameron et al., 1995; Donikian et al., 1996; Gates et al., 1995; Imbsen et al., 1997; Ingham et al., 1996; Jones et al., 1997; Kompfner et al., 1996; Maroney 1996; Prucz et al., 1997; Rodriguez and Inghma, 1996; Schamber et al., 1997; Shirolé and Malik, 1993; Vincent et al., 1997) further confirm that seismically induced damage is likely in steel bridges subjected to large earthquakes and that appropriate measures must be taken to ensure satisfactory seismic performance.

The intent of Article 8.7 is to ensure the ductile response of steel bridges during earthquakes. First, effective load paths must be provided for the entire structure. Following the concept of capacity design, the load effect arising from the inelastic deformations of part of the structure must be properly considered in the design of other elements that are within its load path.

Second, steel substructures must be detailed to ensure stable ductile behavior. Note that the term "substructure" here refers to structural systems exclusive of bearings (Article 8.9) and articulations, which are considered in other Sections. Steel substructures, although few, need ductile detailing to provide satisfactory seismic performance.

Third, considerations for other special ductile systems is introduced, and described in the commentary.

Special consideration may be given to slip-critical connections that may be subjected to cyclic loading. Some researchers have expressed concern that the Poisson effect may cause steel plate thickness to reduce when yielding on net section occurs during seismic response, which may translate into a reduced clamping action on the faying surfaces after the earthquake. This has not been experimentally observed, nor noted in post-earthquake inspections, but the impact of such a phenomenon would be to reduce the slip-resistance of the connection, which may have an impact on fatigue resistance. This impact is believed to be negligible for a Category C detail for finite life, and a Category D detail for infinite life. Design to prevent slip for the Expected Earthquake should be also considered.

C8.7.2 Materials

To ensure that the objective of capacity design is achieved, Grade 250 steel is not permitted for the components expected to respond in a ductile manner. Grade 250 is difficult to obtain and contractors often substitute it with a Grade 345 steel. Furthermore it has a wide range in its expected yield and ultimate strength and very large overstrength factors to cover the anticipated range of property variations. The common practice of dual-certification for rolled shapes, recognized as a problem in the perspective of capacity design following the Northridge earthquake, is now becoming progressively more common also for steel plates. As a result, only Grade 345 steels are allowed within the scope of Article 8.7.2, with a R_y of 1.1.

In those instances when Grade 250 must be used, capacity design must be accomplished assuming a Grade 345 steel (i.e., with a R_y of 1.5 applied to the F_y of 250 Mpa), but R-Factor design and deformation limits shall be checked using Grade 250's yield strength of 250 Mpa.

The use of A992 steel is explicitly permitted. Even though this ASTM grade is currently designated for "shapes for buildings", there is work currently being done to expand applicability to any shapes. ASTM 992 steel, recently developed to ensure good ductile seismic performance, is specified to have both a minimum and maximum guaranteed yield strength, and may be worthy of consideration for ductile energy dissipating systems in steel bridges.

Note that since other steels may be used provided that they are comparable to the approved Grade 345 steels, High Performance Steel (HPS) Grade 345 would be admissible, but not HPS Grade 485 (or higher). This is not a detrimental restriction for HPS steel, as the scope of Article 8.7 encompasses only a few steel members in a typical steel bridge. (Note that, based on very limited experimental data available, it appears that HPS Grade 485 has a lower rotational ductility capacity and may not be suitable for "ductile fuses" in seismic applications).

When other steels are used for energy dissipation purposes, it is the responsibility of the designer to assess the adequacy of material properties available and design accordingly.

Other steel members expected to remain elastic during earthquake shall be made of steels conforming to Article 6.4 of the AASHTO LRFD provisions.

Steel members and weld materials shall have adequate notch toughness to perform in a ductile manner over the range of expected service temperatures. The A709/A709M S84 "Fracture-Critical Material Toughness Testing and Marking" requirement, typically specified when the material is to be utilized in a fracture-critical application as defined by the American Association of State Highway and Transportation Officials (AASHTO), is deemed to be appropriate to provide the level of toughness sought for seismic resistance. For weld metals, note that the AWS D1.5 Bridge Specification requirement for Zone III, familiar to the bridge engineering community, is similar to the 20 ft-lbs at -20F requirement proposed by the SAC Joint Venture for weld metal in welded moment frame connections in building frames."

The capacity design philosophy and the concept of capacity-protected element are defined in Article 4.8.

C8.7.4 Ductile Moment Resisting Frames and Single Column Structures

It is believed that properly detailed fully welded column-to-beam or beam-to-column connections in the moment-resisting frames that would typically be used in bridges can exhibit highly ductile behavior and perform adequately during earthquakes (contrary to what was observed in buildings following Northridge). As a result, strategies to move plastic hinges away from the joints are not required in the Specifications.

However, the engineer may still elect to provide measures (such as haunches at the end of yielding members) to locate plastic hinges some distance away from the welded beam-to-column or column-to-beam joint (FEMA 1995, 1997, 2000).

Although beams, columns and panel zones can all be designed, detailed and braced to undergo severe inelastic straining and absorb energy, the detailing requirements of Article 8.7 address common bridge structures with deep non-compact beams much stiffer flexurally than their supporting steel columns, and favors systems proportioned so that plastic hinges form in the columns. This is consistent with the philosophy adopted for concrete bridges.

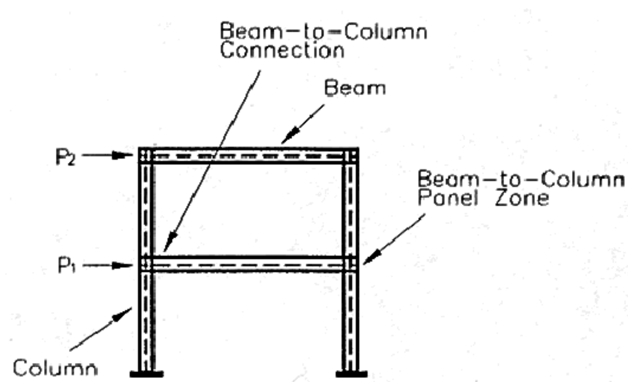


Figure C8.7.4-1 Example of moment frame/bent.

Even though some bridges could be configured and designed to develop stable plastic hinging in beams without loss of structural integrity, the large gravity loads that must be simultaneously be resisted by those beams also make plastic hinging at mid-span likely as part of the plastic collapse mechanism. The resulting deformations can damage the superstructure (diaphragms, deck, etc.).

The special case of multi-tier frames is addressed in Article 8.7.4.4.

C8.7.4.1 Columns

At plastic hinge locations, members absorb energy by undergoing inelastic cyclic bending while maintaining their resistance. Therefore, plastic design rules apply, namely, limitations on width-to-thickness ratios, web-to-flange weld capacity, web shear resistance, lateral support, etc.

Axial load in columns is also restricted to avoid early deterioration of beam-column flexural strengths and ductility when subject to high axial loads. Tests by Popov et al. (1975) showed that W-shaped columns subjected to inelastic cyclic loading suffered sudden failure due to excessive local buckling and strength degradation when the maximum axial compressive load exceeded $0.50A_gF_y$. Tests by Schneider et al. (1992) showed that moment-resisting steel frames with hinging columns suffer rapid strength and stiffness deterioration when the columns are subjected to compressive load equal to approximately $0.25A_gF_y$. Note that most building codes set this limit at $0.30A_gF_y$.

The requirement for lateral support is identical to Equation 6.10.4.1.7-1 of the AASHTO LRFD provisions with a moment (M_l) of zero at one end of the member, but modified to ensure inelastic rotation capacities of at least four times the elastic rotation corresponding to the plastic moment (resulting in a coefficient of 17250 instead of the approximately 25000 that would be obtained for Equation 6.10.4.1.7-1 of the AASHTO LRFD provisions). Consideration of a null moment at one end of the column accounts for changes in location of the inflexion point of the column moment diagram during earthquake response. Figure 10.27 in Bruneau et al. (1997) could be used to develop other unsupported lengths limits.

Built-up columns made of fastened components (bolted, riveted, etc.) are beyond the scope of these specifications.

C8.7.4.2 Beams

Since plastic hinges are not expected to form in beams, beams need not conform to plastic design requirements.

The requirement for beam resistance is consistent with the outlined capacity-design philosophy. The beams should either resist the full elastic loads or be capacity-protected. In the extreme load situation, the capacity-protected beams are required to have nominal resistances of not less than the combined effects corresponding to the plastic hinges in the columns attaining their probable capacity and the probable companion permanent load acting directly on the beams. The columns' probable capacity should account for the overstrength due to higher yield than specified yield and strain hardening effects. The value specified in Article 6.9.2.2 of the AASHTO LRFD provisions, used in conjunction with the resistance factor for steel beams in flexure, ϕ_f, of 1.00, (Article 6.5.4.2 of the AASHTO LRFD provisions) is compatible with the AISC (1997) $1.1R_y$ used with a resistance factor, ϕ, of 0.9 (here R_y is embedded in F_{ye}).

C8.7.4.3 Panel Zones and Connections

The panel zone should either resist the full elastic load (i.e. R=1.0) or be capacity-protected.

Column base connections should also resist the full elastic loads (R=1.0) or be capacity-protected, unless they are designed and detailed to dissipate energy.

Panel zone yielding is not permitted.

There is a concern that doubler plates in panel zones can be an undesirable fatigue detail. For plate-girder sections, it is preferable to specify a thicker web plate if necessary rather than use panel zone doubler plates.

C8.7.4.4 Multi-Tier Frame Bents

Multi-tier frame bents are sometimes used, mostly because they are more rigid transversely than single-tier frame bents. In such multi-tier bents, the intermediate beams are significantly smaller than the top beam as they are not supporting the gravity loads from the superstructure.

As a result, in a multi-tier frame, plastic hinging in the beams may be unavoidable, and desirable, in all but the top beam. In fact, trying to ensure strong-beam weak-column design at all joints in multi-tier bents may have the undesirable effect of concentrating all column plastic hinging in one tier, with greater local ductility demands than otherwise expected in design.

Using capacity design principles, the equations and intent of Article 8.7.4 may be modified by the engineer to achieve column plastic hinging only at the top and base of the column, and plastic hinging at the ends of all intermediate beams, as shown in Figure C8.7.4.4-1.

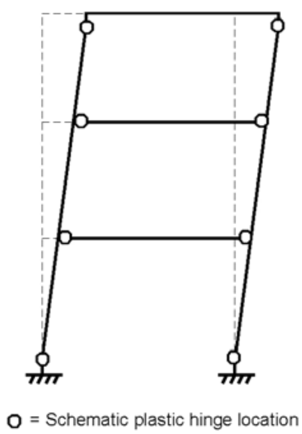


Figure C8.7.4.4-1 Acceptable plastic mechanism for multi-tier bent.

C8.7.5 Ductile Concentrically Braced Frames

Concentrically braced frames are those in which the centerlines of diagonal braces, beams, and columns are approximately concurrent with little or no joint eccentricity. Inelastic straining must take place in bracing members subjected principally to axial load. Compression members can absorb considerable energy by inelastic bending after buckling and in subsequent straightening after load reversal but the amount is small for slender members. Local buckling or buckling of components of built-up members also limits energy absorption.

C8.7.5.1 Bracing Systems

This requirement ensures some redundancy and also similarity between the load-deflection characteristics in the two opposite directions. A significant proportion of the horizontal shear is carried by tension braces so that compression brace buckling will not cause a catastrophic loss in overall horizontal shear capacity. Alternative wording sometimes encountered to express the same intent include:

(a) Diagonal braces shall be oriented such that, at any level in any planar frame, at least 30% of the horizontal shear carried by the bracing system shall be carried by tension braces and at least 30% shall be carried by compression braces.
(b) Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 percent but no more than 70 percent of the total horizontal forced is resisted by tension braces.

This ensures that structural configurations that depend predominantly on the compression resistance of braces (such as case (a) in Figure C8.7.5.1-1) are avoided. Case (b) in that same figure is a better design that meets the above criteria.

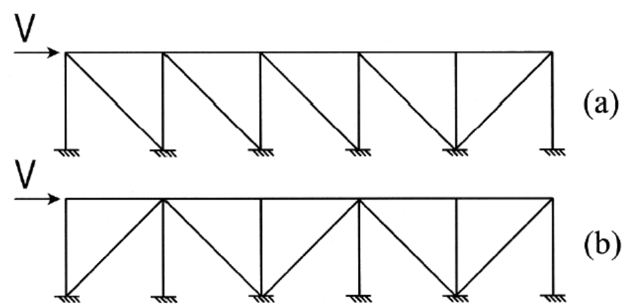


Figure C8.7.5.1-1 Examples of (a) Unacceptable and (b) Acceptable braced bent configurations.

This article also excludes bracing systems that have not exhibited the ductile behavior expected for ductile concentrically braced frames, such as:

(a) Chevron bracing or V-bracing, in which pairs of braces are located either above or below a beam and meet the beam at a single point within the middle half of the span;
(b) K-bracing, in which pairs of braces meet a column on one side near its mid-height; or
(c) Knee-bracing.

C8.7.5.2 Design Requirements for Ductile Bracing Members

Until the late 1990's, for the ductile design of concentrically braced frames in buildings, the slenderness ratio limits for braces were approximately 75% of the value specified here. The philosophy was to design braces to contribute significantly to the total energy dissipation when in compression. Member slenderness ratio was restricted because the energy absorbed by plastic bending of braces in compression diminishes with increased slenderness. To achieve these more stringent KL/r limits, particularly for long braces, designers have almost exclusively used tubes or pipes for the braces. This is unfortunate as these tubular members are most sensitive to rapid local buckling and fracture when subjected to inelastic cyclic loading (in spite of the low width-to-thickness limits prescribed). Recent reviews of this requirement revealed that it may be unnecessary, provided that connections are capable of developing at least the member capacity in tension. This is partly because larger tension brace capacity is obtained when design is governed by the compression brace capacity, and partly because low-cycle fatigue life increases for members having greater KL/r. As a result, seismic provisions for buildings (AISC 1997; CSA 2001) have been revised to permit members having longer KL/r values. The proposed relaxed limits used here are consistent with the new recently adopted philosophy for buildings. The limit for back to back legs of double angle bracing members is increased from the value of Table 8.7.4-1 to $200/\sqrt{F_y}$.

Early local buckling of braces prohibits the braced frames from sustaining many cycles of load reversal. Both laboratory tests and real earthquake observations have confirmed that premature local buckling significantly shortens the fracture life of HSS braces. The more stringent requirement on the b/t ratio for rectangular tubular sections subjected to cyclic loading is based on tests (Tang and Goel, 1987; Uang and Bertero, 1986). The D/t limit for circular sections is identical to that in the AISC plastic design specifications (AISC 1993; Sherman 1976).

C8.7.5.3 Brace Connections

Eccentricities that are normally considered negligible (for example at the ends of bolted or welded angle members) may influence the failure mode of connections subjected to cyclic load (Astaneh, Goel and Hanson, 1986).

A brace which buckles out-of-plane will form a plastic hinge at mid-length and hinges in the gusset plate at each end. When braces attached to a single gusset plate buckle out-of-plane, there is a tendency for the plate to tear if it is restrained by its attachment to the adjacent frame members (Astaneh, Goel and Hanson, 1986). Provision of a clear distance, approximately twice the plate thickness, between the end of the brace and the adjacent members allows the plastic hinge to form in the plate and eliminates the restraint. When in-plane buckling of the brace may occur, ductile rotational behavior should be possible either in the brace or in the joint. Alternatively, the system could be designed to develop hinging in the brace, and the connections shall then be designed to have a flexural strength equal to or greater than the expected flexural strength $1.2R_yM_p$ of the brace about the critical buckling axis.

Buckling of double angle braces (legs back-to-back) about the axis of symmetry leads to transfer of load from one angle to the other, thus imposing significant loading on the stitch fastener (Astaneh, Goel and Hanson, 1986).

C8.7.5.4 Columns, Beams and Other Connections

Columns and beams that participate in the lateral-load-resisting system must also be designed to ensure that a continuous load path can be maintained.

A reduced compressive resistance must be considered for this purpose. This takes into account the fact that, under cyclic loading, the compressive resistance of a bracing member rapidly diminishes. This reduction stabilizes after a few cycles to approximately 30% of the nominal compression capacity.

The unreduced brace compressive resistance must be used if it leads to a more critical condition, as it will be attained in the first cycle. However, redistributed loads resulting from the reduced buckled compressive brace loads must be considered in beams and columns as well as in connections, if it leads to a more critical condition.

Other connections that participate in the lateral-load-resisting system must also be designed to ensure that a continuous load path can be maintained. Therefore,

(a) they must resist the combined load effect corresponding to the bracing connection loads and the permanent loads that they must also transfer; and
(b) they must also resist load effect due to load redistribution following brace yielding or buckling.

C8.7.6 Concentrically Braced Frames with Nominal Ductility

Detailing requirements are relaxed for concentrically braced frames having nominal ductility (a steel substructure having less stringent detailing requirements). They are consequently being designed to a greater force level.

C8.7.6.1 Bracing Systems

This requirement ensures some redundancy and also similarity between the load-deflection characteristics in the two opposite directions. A significant proportion of the horizontal shear is carried by tension braces so that compression brace buckling will not cause a catastrophic loss in overall horizontal shear capacity.

Tension-only systems are bracing systems in which braces are connected at beam-to-column intersections and are designed to resist in tension 100% of the seismic loads.

K-braced frames, in which pairs of braces meet a column near its mid-height, and knee-braced frames shall not be considered in this category.

Systems in which all braces are oriented in the same direction and may be subjected to compression simultaneously shall be avoided.

Analytical and experimental research, as well as observations following past earthquakes, have demonstrated that K-bracing systems are poor dissipators of seismic energy. The members to which such braces are connected can also be adversely affected by the lateral force introduced at the connection point of both braces on that member due to the unequal compression buckling and tension yielding capacities of the braces.

Knee-braced systems in which the columns are subjected to significant bending moments are beyond the scope of this article.

C8.7.6.2 Design Requirements for Nominally Ductile Bracing Members

Nominally ductile braced frames are expected to undergo limited inelastic deformations during earthquakes. Braces yielding in tension are relied upon to provide seismic energy dissipation. While frames with very slender braces (i.e. tension-only designs) are generally undesirable for multistoried frames in buildings, this is mostly because energy dissipation in such frames tend to concentrate in only a few stories, which may result in excessive ductility demands on those braces. However, non-linear inelastic analyses show that satisfactory seismic performance is possible for structures up to 4 stories with tension-only braces, provided that connections are capable of developing at least the member capacity in tension and that columns are continuous over the frame height (CSA 2001). The width-to-thickness ratios for the compression elements of columns can be relaxed for braces having KL/r approaching 200, as members in compression do not yield at that slenderness.

C8.7.6.3 Brace Connections

The additional factor of 1.10 for tension-only bracing systems is to ensure, for the slender members used in this case, that the impact resulting when slack is taken up, does not cause connection failure. Details leading to limited zones of yielding, such as occur at partial joint penetration groove welds should be avoided.

C8.7.6.5 Chevron Braced and V-Braced Systems

Bracing at the beam-brace intersection in chevron and inverted-chevron frames is crucial to prevent lateral torsional buckling of the beam at that location. Effective lateral bracing requires structural elements framing transversely to the frame bent, which may be only possible in 4-column tower piers where horizontal members can be introduced to tie and brace all four faces of the tower pier. Alternatively, lateral bracing could be provided by a connection to the superstructure if proper consideration is given to fatigue and deformation compatibility.

Furthermore, geometry of the braced system must be chosen to preclude beam deformations that could translate into undesirable superstructure damage.

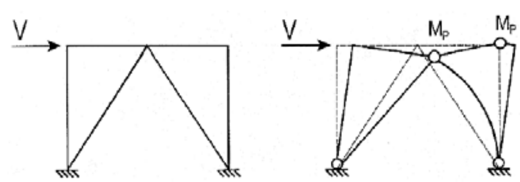


Figure C8.7.6.4-1 Plastic mechanism for a chevron braced bent configuration that would introduce undesirable superstructure damage (unless this bridge has only two girders that are located directly over the columns).

C8.7.7 Concrete Filled Steel Pipes

This article is only applicable to concrete-filled steel pipes without internal reinforcement, and connected in a way that allows development of their full composite strength. It is not applicable to design a concrete-filled steel pipe that relies on internal reinforcement to provide continuity with another structural element, or for which the steel pipe is not continuous or connected in a way that enables it to develop its full yield strength. When used in pile bent, the full composite strength of the plastic hinge located below ground can only be developed if it can be ensured that the concrete fill is present at that location.

Recent research (e.g. Alfawakiri 1997, Bruneau and Marson 1999) demonstrates that the AASHTO equations for the design of concrete-filled steel pipes in combined axial compression and flexure (Articles 6.9.2.2, 6.9.5, and 6.12.2.3.2 of the AASHTO LRFD provisions), provide a very conservative assessment of beam-column strength. Consequently, the calculated strength of concrete-filled steel pipes that could be used as columns in ductile moment resisting frames or pile-bents, could be significantly underestimated. This is not surprising given that these equations together are deemed applicable to a broad range of composite member types and shapes, including concrete-encased steel shapes. While these equations may be perceived as conservative in a non-seismic perspective, an equation that more realistically captures the plastic moment of such columns is essential in a capacity design perspective. Capacity-protected elements must be designed with adequate strength to elastically withstand plastic hinging in the columns. Underestimates of this hinging force translates into under-design of the capacity-protected elements; a column unknowingly stronger than expected will not yield before damage develops in the foundations or at other undesirable locations in the structure. This can be of severe consequences as the capacity protected elements are not detailed to withstand large inelastic deformations. The provisions of Article 8.7.7 are added to prevent this behavior.

Note that for analysis, as implied by Article 6.9.5 of the AASHTO LRFD provisions, flexural stiffness of the composite section can be taken as $E_sI_s + 0.4\ E_cI_c$, where I_c is the gross inertia of the concrete ($\Pi D^4/16$), I_s is the inertia of the steel pipe, and E_s and E_c are respectively the steel and concrete modulus of elasticity.

C8.7.7.1 Combined Axial Compression and Flexure

This equation is known to be reliable up to a maximum slenderness limit D/t of 28000/Fy, underestimating the flexural moment capacity by 1.25 on average. It may significantly overestimate columns strength having greater D/t ratios.

This new equation is only applicable to concrete-filled steel pipes. Other equations may be needed to similarly replace that of Article 6.9.2.2 of the AASHTO LRFD provisions. for other types of composite columns (such as concrete-encased columns).

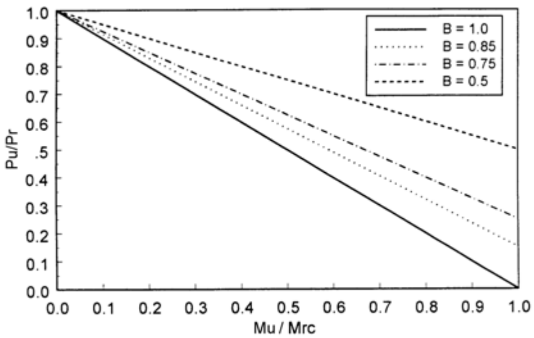


Figure C8.7.7.1-1 Interaction curves for concrete-filled pipes.

C8.7.7.2 Flexural Strength

When using these equations to calculate the forces acting on capacity protected members as a result of plastic hinging of the concrete-filled pipes, F_y should be replaced by F_{ye}, for consistency with the capacity design philosophy.

Figure C8.7.7.2-1 illustrates the geometric parameters used in this Article.

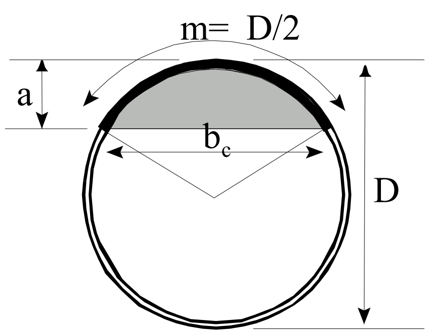
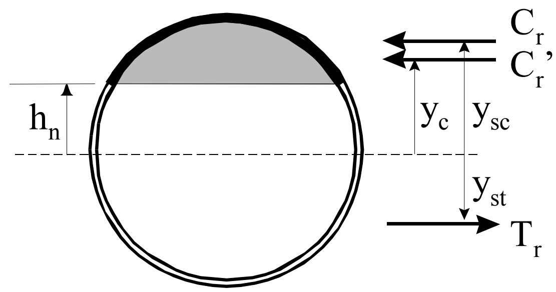


Figure C8.7.7.2-1 Flexure of concrete-filled pipe; shaded area is concrete in compression above the neutral axis.

Moment resistance is calculated assuming the concrete in compression at f'c, and the steel in tension and compression at Fy. The resulting free-body diagram is shown in Figure C8.7.7.2-2, where e is equal to $y_{sc}+y_{st}$, e' is equal to y_c+y_{st}, and y_c is the distance of the concrete compressive force (C_r') from the center of gravity, and y_{st} and y_{sc} are the respective distances of the steel tensile (T_r) and compressive forces (C_r) from the center of gravity.



$$M_{rc} = C_r'(y_c+y_{st}) + C_r\,(y_{sc}+y_{st})$$

Figure C8.7.7.2-2 Free-body diagram used to calculate moment resistance of concrete-filled pipe.

In Method 2, a geometric approximation is made in calculating the area of concrete in compression by subtracting the rectangular shaded area shown in Figure C8.7.7.2-3 from the total area enclosed by the pipe (and dividing the result by 2). Neutral axis is at height h_n.

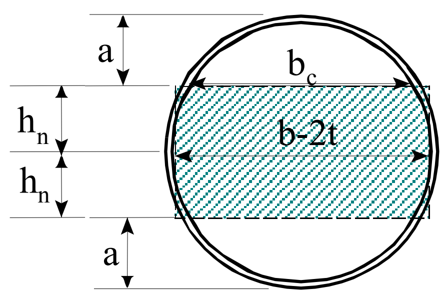


Figure C8.7.7.2-3 Flexure of concrete-filled pipe – illustrates approximation made in Method 2.

Method 2 (using approximate geometry) gives smaller moments compared to Method 1 (exact geometry). The requirement to increase the calculated moment by 10% for capacity design when using the approximate method was established from the ratio of the moment calculated by both methods for a D/t of 10. That ratio decreases as D/t increases.

C8.7.7.3 Beams and Connections

Recent experimental work by Bruneau and Marson (1999), Shama et al. (2001), Azizinamini et al. (1999), provide examples of full fixity connection details. Note that, in some instances, full fixity may not be needed at both ends of columns. Concrete-filled steel pipes, when used in pile bents, only require full moment connection at the pile-cap.

C8.7.8 Other Systems

Article 8.7.8, Other Systems, contains systems less familiar to bridge engineers. Eccentrically braced substructures are included in this section partly for that reason, but also because most configurations of this system would introduce beam deformations that are undesirable in bridges as this could translate into superstructure damage. Furthermore, bracing of the links may be a difficult design issue that requires special consideration in bridge bents.

The engineer must take the necessary steps to ensure that special systems will provide a level of safety comparable to that provided in these Specifications. This may require review of published research results, observed performance in past earthquakes, and/or special investigations.

C8.7.8.1 Ductile Eccentrically Braced Frames

Note that the scope of 8.7.8.1 is for eccentrically braced frames used as ductile substructure, not as part of ductile diaphragms.

Eccentrically braced frames have been extensively tested and implemented in numerous buildings, but, at the time of this writing, few new bridges have been built relying on shear links for seismic energy dissipation. An obvious difficulty in bridge applications arises because the eccentric link cannot be easily laterally braced to prevent movement out of the plane of the braced bent. Nonetheless, the bents of the Richmond-San Raphael bridge near San Francisco have been retrofitted using eccentrically braced frames. For that bridge, multiple adjacent frames were used to be able to provide proper bracing of the shear links. Large scale testing was conducted to validate that retrofit concept (Vincent 1996; Itani et al, 1998b). Furthermore, the tower of the new east bay crossing of the Bay Bridge between San Francisco and Oakland is connected by shear links, albeit not in an eccentrically braced frame configuration (Tang et al., 2000).

While effective eccentrically braced bents are possible, only details that have been tested with the same lateral bracing considerations as in the prototype must be used. Other details must be experimentally validated. Note that size effects have not been fully investigated. Although it is preferable to use links of sizes no greater than those validated by full-scale tests, in some instances, this may not be possible.

Extensive detailing requirements are not provided within these specifications. However, the engineer could follow the detailing practice used for buildings, modified to address the above concerns regarding lateral bracing.

The scope of this article is restricted to eccentrically braced frame of split-V configuration. Eccentrically braced frames configurations in which the ductile link is adjacent to a beam-column connection are prohibited, unless it can be demonstrated by tests of specimens of size greater or equal to the prototype that the connection can develop the required strength and hysteretic ductility.

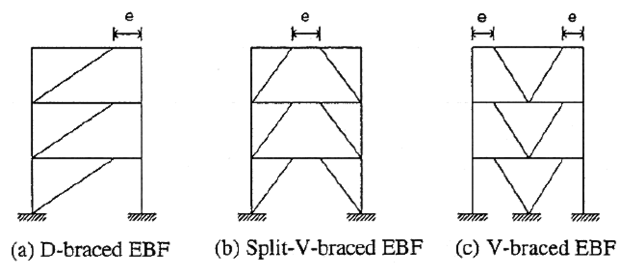


Figure C8.7.8.1-1 Eccentrically braces frames configurations, the scope of C6.15.5.1 being restricted to split-V configuration (case b).

Furthermore, geometry of the eccentrically braced system must be chosen to preclude beam deformations that could translate into undesirable superstructure damage,. As such, the configurations shown in Figure C8.7.8.1-1 would introduce undesirable superstructure damage, unless this bridge has only two girders that are located directly over the columns. In most cases, alternative configurations would be required.

For eccentrically braced frames, all references to “inelastic hinging of the column” in other seismic requirements elsewhere in the Specifications should be interpreted as “yielding of the eccentric link”.

C8.7.8.2 Ductile End-Diaphragm in Slab-on-Girder Bridge

The ductile diaphragm strategy is not effective when the substructure is significantly more flexible than the superstructure. This is addressed by Article 8.7.8.2. Bridges having wide piers, wall-piers, or other substructure elements of similar limited ductility, would be good candidates for the implementation of the ductile diaphragm system. In these examples, the ductile diaphragms could also be designed to yield instead of the bridge piles, thus preventing the development of damage below ground level where it cannot be inspected following an earthquake.

The contribution of girders can be significant and cannot be neglected, as indicated in Article 8.7.8.2. For that reason, ductile diaphragm are generally more effective in longer span bridges, and may be of limited benefit for short span bridges.

Note that the inertia forces attributable to the mass of the pier-cap will be resisted by the substructure, in spite of the presence of ductile diaphragms. Refined analyses should consider this condition if that mass is a significant portion of the total superstructure mass.

For ductile end-diaphragms, all references to "inelastic hinging of the column" in other seismic requirements elsewhere in the Specifications should be interpreted as "yielding of the ductile diaphragm".

A detailed procedure for the design of ductile diaphragms is presented in Appendix E, along with illustrations of systems that would satisfy the restrictions of Article 8.7.8.2.

C8.7.8.3 Ductile End Diaphragms in Deck Truss Bridges

Articles 8.7.8.2 and 8.7.8.3 share much conceptual similarities, but seismic forces in deck-trusses follow a more complex and redundant load-path. This requires the use of ductile diaphragms vertically over the supports as well as horizontally in the last lower horizontal cross-frame before each support.

For ductile end-diaphragms, all references to "inelastic hinging of the column" in other seismic requirements elsewhere in the Specifications should be interpreted as "yielding of the ductile diaphragm".

Further research may allow to relax the limits imposed by Article 8.7.8.3.

A detailed procedure for the design of ductile diaphragms is presented in Appendix F.

C8.7.8.4 Other Systems

Note that many other "special systems" may emerge in the future, such as friction-braced frames, shock transmission units, other approaches of superstructure plastic hinging, marine bumpers etc.

C8.7.9 Plastic Rotational Capacities

A moment-curvature analysis based on strain compatibility and non-linear stress-strain relations can be used to determine plastic limit states. From this, a rational analysis is used to establish the rotational capacity of plastic hinges.

C8.7.9.3 In Ground Hinges

In-ground hinges are necessary for certain types of bridge substructures. These may include, but not restricted to:

- Pile bents
- Pile foundations with strong pier walls
- Drilled shafts
- Piled foundations with oversized columns

It is necessary to restrict these plastic hinge rotations in order to limit plastic strains. This limit is expected to reduce plastic strains to less than 10 percent of their above-ground counterpart (with θ_p=0.035 radians), due to the increased plastic hinge length of in-ground hinges.

C8.8 REINFORCED CONCRETE DESIGN REQUIREMENTS

C8.8.1 General

High strength reinforcement reduces congestion and cost as demonstrated by Mander and Cheng (1999), and Dutta, Mander and Kokorina, (1999). However it is important to ensure that the cyclic fatigue life is not inferior when compared to ordinary mild steel reinforcing bars. Mander, Panthaki, and Kasalanati, (1994) have shown that modern high alloy prestressing threadbar steels can have sufficient ductility to justify their use in seismic design.

The *Modulus of Toughness* is defined as the area beneath the monotonic tensile stress-strain curve from initial loading (zero stress) to fracture.

Bridge Designers working with sites subjected to Seismic Hazard Levels III and IV are encouraged to avail themselves of current research reports and other literature to augment these Specifications.

The 1989 Loma Prieta and 1994 Northridge earthquakes confirmed the vulnerability of columns with inadequate transverse reinforcement and inadequate anchorage of longitudinal reinforcement. Also of concern:

- Lack of adequate reinforcement for positive moments that may occur in the superstructure over monolithic supports when the structure is subjected to longitudinal dynamic loads;
- Lack of adequate shear strength in joints between columns and bent caps under transverse dynamic loads; and
- Inadequate reinforcement for torsion, particularly in outrigger-type bent caps.
- Inadequate transverse reinforcement for shear and restraint against global buckling of longitudinal bars ("bird caging")

The purpose of the additional design requirements of this article is to increase the probability that the design of the components of a bridge are consistent with the principles of "Capacity Design", especially for bridges located in Seismic Hazard Levels II to IV, and that the potential for failures observed in past earthquakes is minimized. The additional column design requirements of this article for bridges located in Seismic Hazard Levels III and IV are to ensure that a column is provided with reasonable ductility and is forced to yield in flexure and that the potential for a shear, compression failure due to longitudinal bar buckling, buckling, or loss of anchorage mode of failure is minimized. See also Articles 3.3 and 4.8 for further explanation. The actual ductility demand on a column or pier is a complex function of a number of variables, including:

- Earthquake characteristics, including duration, frequency content and near field (pulse) effects.
- Design force level,
- Periods of vibration of the bridge,
- Shape of the inelastic hysteresis loop of the columns, and hence effective hysteretic damping.
- Elastic damping coefficient,
- Contributions of foundation and soil conditions to structural flexibility, and
- Spread of plasticity (plastic hinge length) in the column.

The damage potential of a column is also related to the ratio of the duration of strong motion shaking to the natural period of vibration of the bridge. This ratio will be an indicator of the low cycle fatigue demand on the concrete column hinge zones.

C8.8.2 Column Requirements

The definition of a column in this article is provided as a guideline to differentiate between the additional design requirements for a wall-type pier and the requirements for a column. If a column or pier is above or below the recommended criterion, it may be considered to be a column or a pier, provided that the appropriate R-Factor of Article 4.7 and the appropriate requirements of either Articles 8.8.2 or 8.8.3 are used. For columns with an aspect ratio less than 2.5, the forces resulting from plastic hinging will generally exceed the elastic design forces; consequently, the forces of Article 8.8.3 would not be applicable.

Certain oversize columns exist for architectural/aesthetic reasons. These columns, if fully reinforced, place excessive moment and/or shear demands on adjoining elements. The designer should strive to "structurally isolate" those architectural elements that do not form part of the primary energy dissipation system that are located either within or in close proximity to plastic hinge zones. Nevertheless, the architectural elements must remain serviceable throughout the life of the structure. For this reason, minimum steel for temperature and shrinkage should be provided. Note that, when architectural flares are not isolated, Article 4.8 requires that the design shear force for a flared column be the worst case calculated using the overstrength moment of the oversized flare or the shear generated by a plastic hinge at the bottom of the flare.

C8.8.2.1 Longitudinal Reinforcement

This requirement is intended to apply to the full section of the columns. The 0.8 percent lower limit on the column reinforcement reflects the traditional concern for the effect of time-dependent deformations as well as the desire to avoid a sizable difference between the flexural cracking and yield moments. The 4 percent maximum ratio is to avoid congestion and extensive shrinkage cracking and to permit anchorage of the longitudinal steel, but most importantly, the less the amount of longitudinal reinforcement, the greater the ductility of the column.

C8.8.2.2 Flexural Resistance

Columns are required to be designed biaxially and to be investigated for both the minimum and maximum axial forces. Resistance factors of unity may be used wherever moments and axial loads are derived from a plastic mechanism.

C8.8.2.3 Column Shear and Transverse Reinforcement

The implicit method is conservative and is most appropriate when a shear demand has not been calculated, e.g., SDR 2 and piles. The explicit method should result in less reinforcement and is recommended if the shear demand is available.

This implicit shear detailing approach assumes that

$$\phi V_u = V_c + V_p + V_s \geq \frac{\Lambda M_p^o}{H_c}$$

in which $V_c = 0$ (the contribution of shear carried by the concrete tensile section). This shear demand at plastic overstrength (M_p^o) is implicitly resisted by arch action (V_p) which is carried by a corner-to-corner diagonal strut in the concrete, and truss action (V_s) which is resisted by the transverse reinforcement. The overstrength demand for the transverse steel comes solely from the presence of the longitudinal reinforcement. It is for this reason the transverse steel (ρ_v) is directly proportional to the longitudinal steel (ρ_t). Thus, if steel congestion results for a chosen column size, one viable solution is to enlarge the column and reduce the longitudinal steel volume.

For a derivation of the implicit shear detailing approach, refer to the recent research by Dutta and Mander (1998).

The requirements for shear outside of the hinge zones assumes the concrete is capable of sustaining a concrete stress of $v_c = 0.17\sqrt{f_c'}\cot\theta$.

The basis of equation (8.8.2.3-5) follows

Shear in end zones = shear outside end zones

$$V_s = V_s^* + V_c$$

where V_s^* = shear carried by the transverse steel outside the plastic hinge zone. Expanding both sides gives

$$\rho_v A_v f_{yh} \cot\theta = \rho_v^* A_v f_{yh} \cot\theta + 0.17\sqrt{f_c'}\cot\theta A_v$$

Solving for ρ_v^*, the required amount of transverse reinforcement outside the potential plastic hinge zone, gives equation (8.8.2.3-5) Note that if ρ_v^* is negative, this means the concrete alone is theoretically adequate for strength, although the minimum steel is still required if this occurs.

The shear strength model is based on the concept that the total shear strength is given by the following design equation:

$$V_u < V_s + V_p + V_c$$

The concrete tensile contribution to shear, Vc, is assumed to significantly diminish under high ductilities and cyclic loading.

The requirements of this article are intended to avoid column shear failure by using the principles of "capacity protection". The design shear force is specified as a result of the actual longitudinal steel provided, regardless of the design forces. This requirement is necessary because of the potential for superstructure collapse if a column fails in shear.

A column may yield in either the longitudinal or transverse direction. The shear force corresponding to the maximum shear developed in either direction for noncircular columns should be used for the determination of the transverse reinforcement.

For a noncircular pile, this provision may be applied by substituting the larger cross-sectional dimension for the diameter.

As a starting point for initial design, assume $\theta = 35^\circ$. The actual crack angle should be estimated based on the provided transverse reinforcement using equation 8.8.2.3-14. From this the shear strength should be checked based on the provided steel.

The Explicit shear approach defined herein is similar to the shear model of Priestley, Verma and Xiao (1994). Based on a survey of empirical observations, Priestley et al. recommended that the crack angle be taken as $\theta = 35^\circ$ and 30^o for design and analysis, respectively.

The crack angle computed in equation 8.8.2.3-14 is more general. The associated theory is based on research by Kim and Mander (1999). In their approach an energy minimization of shear-flexure deflections was used on a truss model of a beam-column element to find an analytical expression for the crack angle. This theoretical crack angle equation was then validated against a wide variety of experimental observations.

C8.8.2.4 Transverse Reinforcement for Confinement at Plastic Hinges

Plastic hinge regions are generally located at the top and bottom of columns and pile bents. should govern; these requirements are not in addition to those of Article 8.8.2.3.

These equations ensure that the concrete is adequately confined so that the transverse hoops will not prematurely fracture as a result of the plastic work done on the critical column section. For typical bridge columns with low levels of axial load, these equations rarely govern, but must be checked. The equations were developed by Dutta and Mander (1998), with experiments demonstrating that they work well for both regular mild steel spirals as well as high strength steel in the form of wire rope (see Dutta et al, 1999). Note the latter should not be used for hoops, ties or stirrups with bent hooks.

Loss of concrete cover in the plastic hinge zone as a result of spalling requires careful detailing of the confining steel. It is clearly inadequate to simply lap the spiral reinforcement. If the concrete cover spalls, the spiral will be able to unwind. Similarly, rectangular hoops should be anchored by bending ends back into the core.

Figures C8.8.2.4-1 through C8.8.2.4.-4 illustrate the use of Equations 8.8.2.4-1 and -2. The required total area of hoop reinforcement should be determined for both principal axes of a rectangular or oblong column, and the greater value should be used.

While these Specifications allow the use of either spirals, hoops or ties for transverse column reinforcement, the use of spirals is recommended as the more effective and economical solution. Where more than one spiral cage is used to confine an oblong column core, the spirals should be interlocked with longitudinal bars as shown in Figure C8.8.2.4-3. Spacing of longitudinal bars of a maximum of 200 mm center-to-center is also recommended to help confine the column core.

Examples of transverse column reinforcement are shown herein.

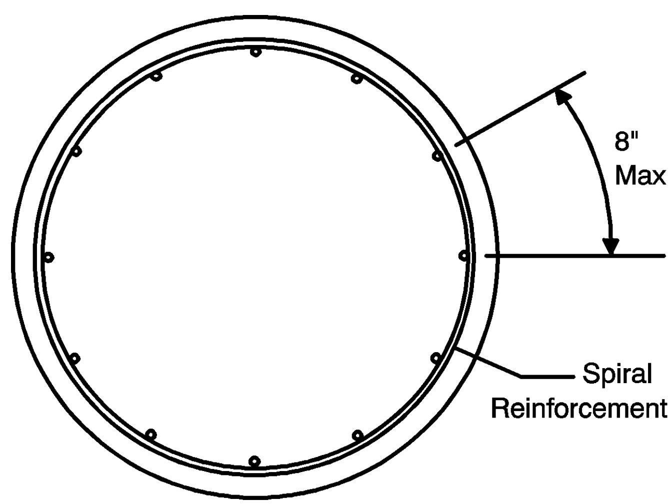


Figure C.8.8.2.4-1 Single Spiral

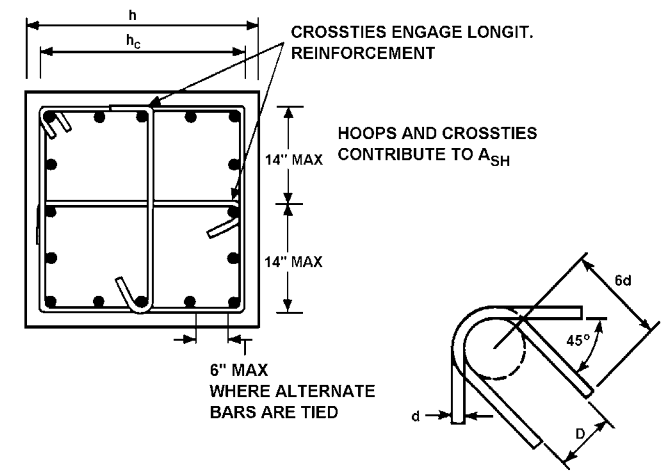


Figure C8.8.2.4-2 Column Tie Details

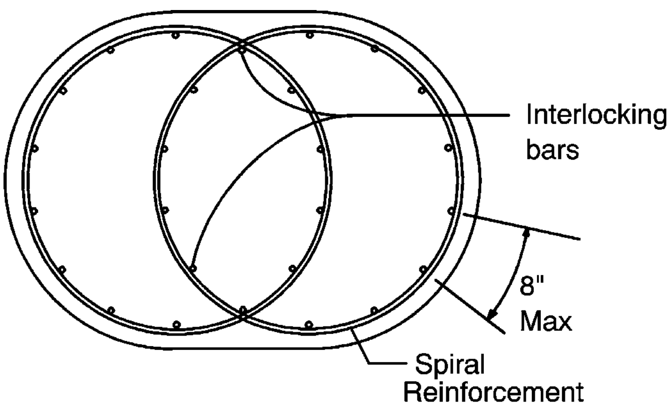


Figure C8.8.2.4-3 Column Interlocking Spiral Details

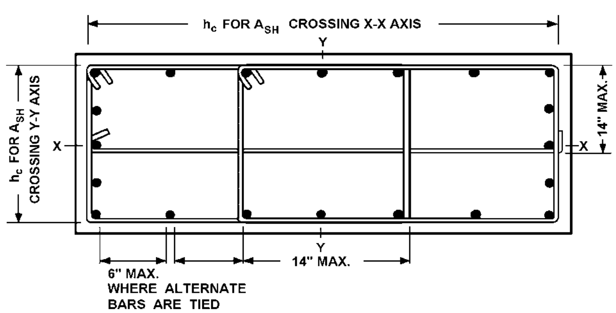


Figure C8.8.2.4-4 Column Tie Details

C8.8.2.5 Transverse Reinforcement for Longitudinal Bar Restraint in Plastic Hinges

Longitudinal reinforcing bars in potential plastic hinge zones may be highly strained in compression to the extent they may buckle. Buckling may either be

(a) local between two successive hoop sets or spirals, or
(b) global and extend over several hoop sets or spirals.

Condition (a) is prevented by using the maximum vertical spacing of transverse reinforcement given by Equation 7.8.2.5-1 of the Specifications.

Although research has been conducted to determine the amount of transverse reinforcement required to prevent condition (b), this research has not been fully peer reviewed, and thus has not been included as part of the Specifications. However, designers should not ignore the possibility of condition (b) and should take steps to prevent it.

The following tentative criteria for transverse reinforcement to prevent condition (b) have been proposed:

(i) for circular sections confined by spirals or circular hoops

$$\rho_s = 0.016\left(\frac{D}{s}\right)\left(\frac{s}{d_b}\right)\rho_t \frac{f_y}{f_{yh}} \qquad (7.8.2.5\text{-}2)$$

(ii) for rectangular sections confined by transverse hoops and/or cross ties the area of the cross tie or hoop legs (A_{bh}) shall be:

$$A_{bh} = 0.09 A_b \frac{f_y}{f_{yh}} \qquad (7.8.2.5\text{-}3)$$

where

ρ_s = ratio of transverse reinforcement

$$\rho_s = \frac{4A_{bh}}{sD'}$$

D = diameter of circular column

d_b = diameter of longitudinal reinforcing bars being restrained by circular hoop or spiral
A_b = area of longitudinal reinforcing bars being restrained by rectilinear hoops and/or cross ties
A_{bh} = bar area of the transverse hoops or ties restraining the longitudinal steel
ρ_t = volumetric ratio of longitudinal reinforcement
f_y = yield stress of the longitudinal reinforcement
f_{yh} = yield stress of the transverse reinforcing bars

It should be noted that trial applications have shown that the above equations result in excessive transverse reinforcement in some cases. This is usually associated with high amounts of column longitudinal reinforcement, and so it may be prudent for a designer to limit the volumetric ratio of longitudinal reinforcement.

Criteria (i) and (ii) are intended to ensure that the yield capacity of the longitudinal reinforcement is maintained. This is a life-safety requirement. If global buckling of the longitudinal reinforcing is to be inhibited to ensure post-earthquake repairability, then it has been proposed that the following be used:

$$\rho_s = 0.024\left(\frac{D}{d_b}\right)\rho_t \frac{f_y}{f_y h}$$

and

$$A_{bh} = 0.25 A_b \frac{f_y}{f_y h}$$

Criteria (i) and the above recommendation for post-earthquake repairability may lead to congestion of hoops/spirals in circular columns with large amounts of longitudinal reinforcement. One way to overcome this is to use wire rope or prestressing strand as transverse reinforcement with a high yield strain. However, this is another reason for not mandating the global anti-buckling criteria since it would require a major change in construction practice that needs to be more thoroughly evaluated from the standpoint of constructibility.

An alternate approach to relieve transverse reinforcement congestion arising from these antibuckling requirements is to use two concentric rings of longitudinal steel. The antibuckling requirements need only apply to the outer ring of longitudinal bars.

C8.8.2.6 Spacing for Transverse Reinforcement for Confinement and Longitudinal Bar Restraint

This requirement ensures all inelastic portions of the column are protected by confining steel.

C8.8.2.7 Splices

It is often desirable to lap longitudinal reinforcement with dowels at the column base. This is undesirable for seismic performance because:

- The splice occurs in a potential plastic hinge region where requirements for bond is critical, and
- Lapping the main reinforcement will tend to concentrate plastic deformation close to the base and reduce the effective plastic hinge length as a result of stiffening of the column over the lapping region. This may result in a severe local curvature demand.

C8.8.2.8 Flexural Overstrength

The simplified method for calculating an overstrength moment-axial load interaction diagram (Mander, et. al, 1997) involves a parabolic curve fit to (M_{bo}, P_b) and (0, P_{to}) given by Equation C8.8.2.8-1.

$$\frac{M_{po}}{f'_c A_g D} = \left(\frac{M_{bo}}{f'_c A_g D}\right)\left[1 - \left(\frac{\frac{P_e}{f'_c A_g} - \frac{P_b}{f'_c A_g}}{\frac{P_{to}}{f'_c A_g} - \frac{P_b}{f'_c A_g}}\right)^2\right] \quad \text{(C8.8.2.8-1)}$$

where:

$\frac{P_e}{f'_c\,Ag}$ = axial stress ratio on the column based on gravity load and seismic (framing) actions

$\frac{P_{to}}{f_{c'}\,A_g} = -\rho_t \frac{f_{su}}{f_{c'}}$ = normalized axial tensile capacity of the column

$\frac{P_b}{f_{c'}\,A_g} = 0.425\beta_1$ = normalized axial load capacity at the maximum nominal (balanced) moment on the section where β_1 = stress block factor (≤ 0.85)

$$\frac{M_{bo}}{f_c' A_g D} = \left(K_{shape}\rho_t \frac{f_{su}}{f_c'}\frac{D'}{D} + \frac{P_b}{f_c' A_g}\frac{1-\kappa_o}{2} \right) \qquad (8.8.2.8\text{-}2)$$

D' = pitch circle diameter of the reinforcement in a circular section, or the out-to-out dimension of the reinforcement in a rectangular section, this generally may be assumed as $D' = 0.8D$.

f_{su} = ultimate tensile strength of the longitudinal reinforcement.

K_{shape} should be taken defined in Article 8.8.2.3.

κ_o = a factor related to the stress block centroid which should be taken as 0.6 and 0.5 for circular and rectangular sections, respectively.

C8.8.3 Limited Ductility Requirements for Wall Type Piers

The requirements of this article are based on limited data available on the behavior of piers in the inelastic range. Consequently, the R-Factor of 2.0 for piers is based on the assumption of minimal inelastic behavior.

The requirement that $\rho_v \geq \rho_h$ is intended to avoid the possibility of having inadequate web reinforcement in piers which are short in comparison to their height. Splices should be staggered in an effort to avoid weak sections.

C8.8.4 Moment Resisting Connection Between Members (Column/Beam and Column/Footing Joints)

C8.8.4.1 Implicit Approach: Direct Design

Shear steel will often govern in connections due to the increased shear demand at flexural overstrength arising from a smaller shear span within the joint compared to the columns framing into the connection. If this results in considerable congestion, particularly when large volumes of longitudinal steel exist, then design method 2 might give some relief. This is because methods 2 permits some of the joint reinforcement to be placed outside the joint in the adjacent cap beam.

C8.8.4.2 Explicit Approach: Detailed Design

The designer may consider the following means to improve constructability:

- prestressing the joint as a means of reducing reinforcing steel,
- placing vertical shear reinforcement within the joint and/or in the cap beam adjacent to the joint region.

C8.8.4.2.1 Design Forces and Applied Stresses

The stresses f_h and f_v in Equations 8.8.4.2-1 and 8.8.4.2-2 are nominal compression stresses in the horizontal and vertical directions, respectively. In a typical joint f_v is provided by the column axial force P_e. An average stress at midheight of the cap beam, or mid-depth of the footing, should be used, assuming a 45-degree spread away from the boundaries of the column in all directions. The horizontal axial stress f_h is based on the mean axial force at the center of the joint, including effects of cap beam prestress, if present.

The joint shear stress v_{hv} can be estimated with adequate accuracy from the expression

$$v_{hv} = \frac{M_p}{h_b h_c b_{ji}} \qquad (8.8.4.2\text{-}1)$$

where

M_p = the maximum plastic moment

h_b = the cap beam or footing depth

h_c = the column lateral dimension in the direction considered (i.e., $h_c = D$ for a circular column)

b_{je} = the effective joint width, found using a 45-degree pread from the column boundaries.

Figures C8.8.4.2-1 (Priestley, Seible and Calvi, 1996) clarify the quantities to be used in this calculation.

C8.8.4.2.2 Maximum Required Horizontal Reinforcement

The need to include spiral reinforcement to aid in joint force transfer has become obvious as a result of the poor performance of moment-resisting connections in recent earthquakes and in

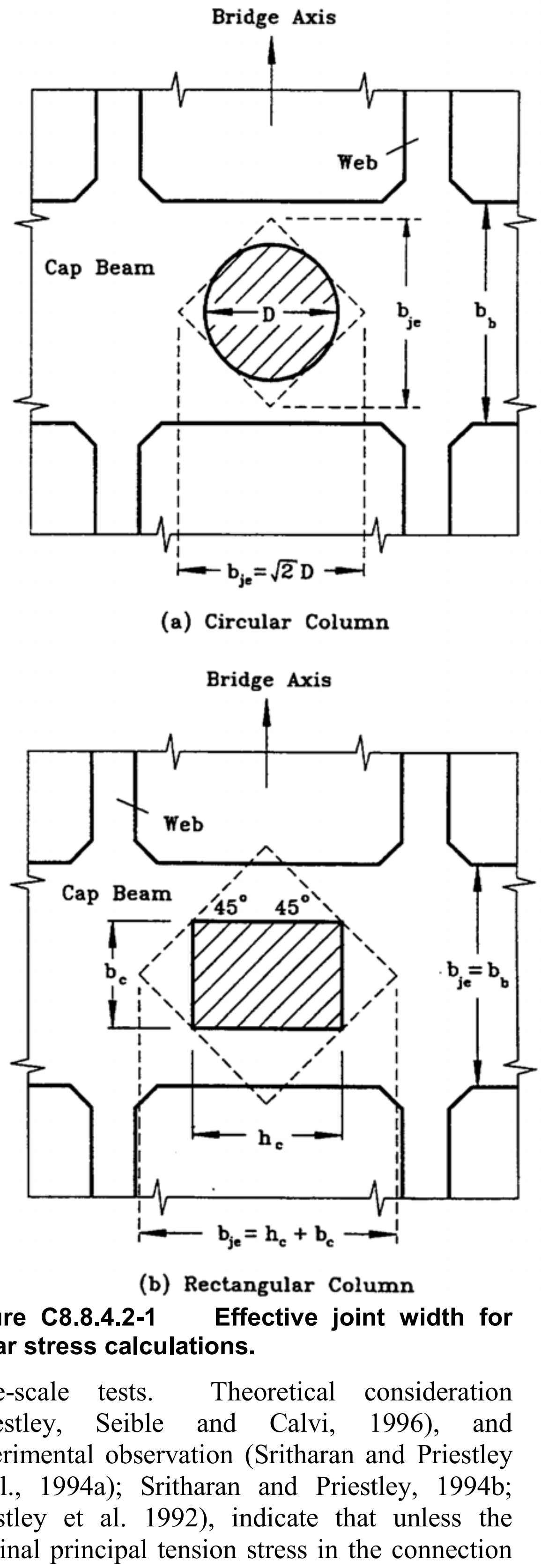


Figure C8.8.4.2-1 Effective joint width for shear stress calculations.

large-scale tests. Theoretical consideration (Priestley, Seible and Calvi, 1996), and experimental observation (Sritharan and Priestley et al., 1994a); Sritharan and Priestley, 1994b; Preistley et al. 1992), indicate that unless the nominal principal tension stress in the connection (join region) exceeds $0.29\sqrt{f_c'}$ MPa, diagonal cracking in the connection will be minimal. Equation 8.8.4.2-3 requires placement of sufficient hoop reinforcement to carry 50 percent of the tensile force at $0.29\sqrt{f_c'}$ MPa, nominal tensile stress, resolved into the horizontal plane. This is minimum level of reinforcement.

C8.8.4.2.3 Maximum Allowable Compression Stresses

The principal compression stress in a connection is limited to $0.25\,f_c'$. This limits the shear stress to less than $0.25\,f_c'$. It is felt that the level of nominal principal compression stress is a better indicator of propensity for joint crushing than is the joint shear stress.

C8.8.4.3 Reinforcement for Joint Force Transfer

C8.8.4.3.1 Acceptable Reinforcement Details

A "rational" design is required for joint reinforcement when principal tension stress levels exceed $0.29\sqrt{f_c'}$ MPa. The amounts of reinforcement required are based on the mechanism shown in Figure C8.8.4.3-1 which primarily uses external reinforcement for joint resistance to reduce joint congestion.

C8.8.4.3.2 Vertical Reinforcement

Stirrups

Figure C8.8.4.3-1 is intended to clarify this clause. A_{ST} is the total area of column reinforcement anchored in the joint. Reinforcement A_{jv} is required to provide the tie force T_s resisting the vertical component of strut D2 in Figure C8.8.4.3-1. This reinforcement should be placed close to the column cage for maximum efficiency.

Clamping Reinforcement

In addition, it will be recognized that the cap beam top reinforcement or footing bottom reinforcement may have severe bond demands, since stress levels may change from close to tensile yield on one side of the joint to significant levels of compression stress on the other side. The required $0.08\,A_{ST}$ vertical ties inside the joint are intended to help provide this bond transfer by clamping the cap-beam rebar across possible splitting cracks. Similar restraint may be required for superstructure top longitudinal rebar. Cap beam widths one foot greater than column diameter are encouraged so that the joint shear reinforcement is effective.

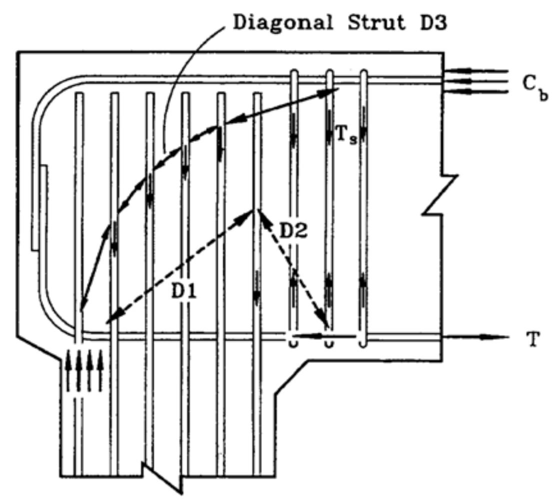


Figure C8.8.4.3-1 External Vertical Joint Reinforcement for Joint Force Transfer.

When the cap beam and/or superstructure is prestressed, the bond demands will be much less severe and the clamping requirement can be relaxed. It can also be shown theoretically (Priestley, Seible and Calvi, 1996) that the volumetric ratio of hoop reinforcement can be proportionately reduced to zero as the prestress force approaches $0.25T_c$.

Figure C8.8.4.3-2 shows each of the areas within which the reinforcement required by this clause must be placed. For an internal column of a multi-column bent, there will be four such areas, overlapping, as shown in Figure C8.8.4.3-2a). For an exterior column of a multi-column bent, there will be three such areas (Figure C8.8.4.3-2b). For a single-column bent with monolithic column/cap beam connection, there will be two such areas corresponding to longitudinal response (Figure

C8.8.4.3-2c). Where these areas overlap, vertical joint reinforcement within the overlapping areas may be considered effective for both directions of response. Where shear reinforcement exists within a given area and is not fully utilized for shear resistance in the direction of response considered, that portion not needed for shear resistance may be considered to be vertical joint reinforcement. Since cap beam shear reinforcement is normally dictated by conditions causing cap beam negative moment (gravity and seismic shear are additive) while the external joint reinforcement discussed in this section applies to cap beam positive moment (when gravity and seismic shear are in opposition), it is normal to find that a considerable portion of existing cap beam shear reinforcement adjacent to the joint can be utilized.

C8.8.4.3.3 Horizontal Reinforcement

Additional cap-beam bottom reinforcement of area $0.08A_{ST}$ is required to provide the horizontal resistance of the strut D2 in Figure C8.8.4.3-1.

Special care is needed for knee joints as represented by Figure C8.8.4.3-2(b). For moment tending to close the joint, force transfer must be provided between the top cap beam reinforcement and the column outer reinforcement. When the cap beam does not extend significantly past the column, this is best effected by making the cap beam top and bottom reinforcement into a continuous loop outside the column cage, as shown in Figure C8.8.4.3-1.

If a cap-beam cantilever is provided, with cap-beam reinforcement passing beyond the joint, additional vertical shear reinforcement outside the joint, as for Figure C8.8.4.3-2, will be required.

Moment-resisting connections designed according to these requirements have performed well in experiments (Seible et al., 1994; Sritharran and Priestley, 1994a; Sritharan and Priestley, 1994b).

This reinforcement may be omitted in prestressed or partially prestressed cap beams if the prestressed design force is increased by the amount needed to provide an equivalent increase in cap-beam moment capacity to that provided by this reinforcement.

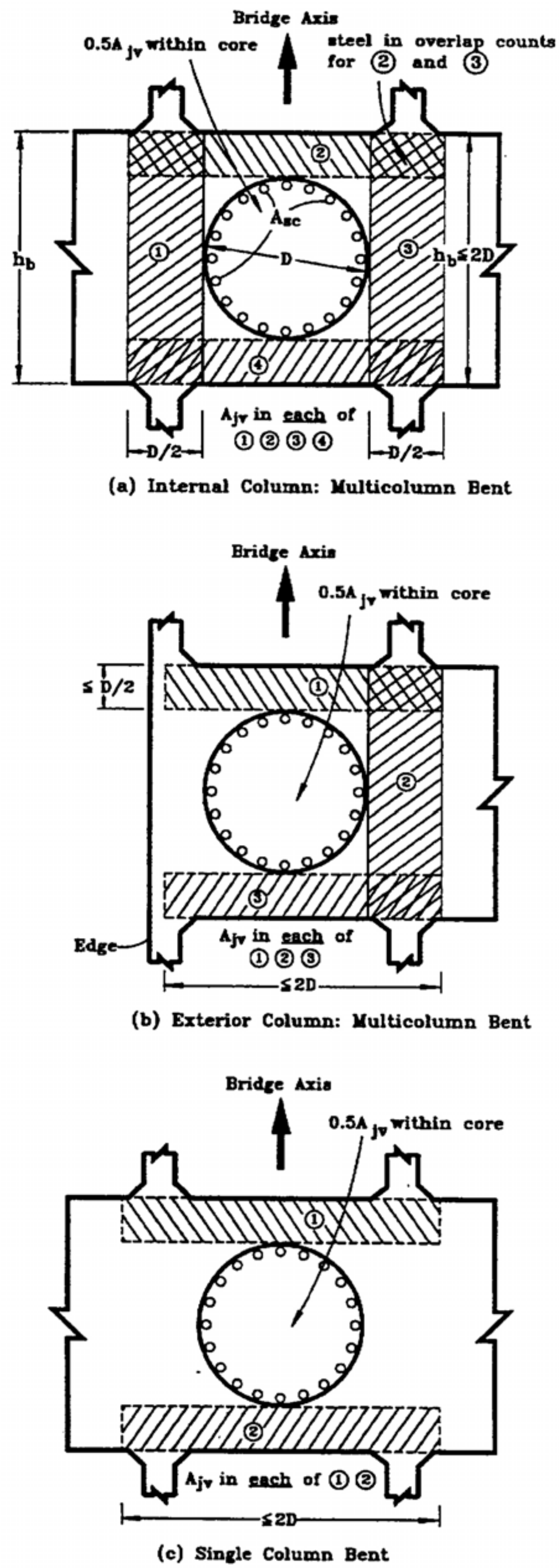


Figure C8.8.4.3-2 Locations for Vertical Joint Reinforcement.

C8.8.4.3.4 Hoop or Spiral Reinforcement

The hoop or spiral reinforcement of Equation 8.8.4.3-1 is required to provide adequate confinement of the joint, and to resist the net outward thrust of struts D1 and D2 in Figure C8.8.4.3-1.

C8.8.4.4 Footing Strength

Under extreme seismic loading, it is common for the footing to be subjected to positive moments on one side of the column and negative moments on the other. In this case, shear lag considerations show that it is unrealistic to expect footing reinforcement at lateral distances greater than the footing effective depth to effectively participate in footing flexural strength. Tests on footings (Xiao et al., 1994) have shown that a footing effective width complying with this clause will produce a good prediction of maximum footing reinforcement stress. If a larger effective width is adopted in design, shear lag effects will result in large inelastic strains developing in the footing reinforcement adjacent to the column. This may reduce the shear strength of the footing and jeopardize the footing joint force transfer mechanisms. Since the reinforcement outside the effective width is considered ineffective for flexural resistance, it is permissible to reduce the reinforcement ratio in such regions to 50 percent of that within the effective width unless more reinforcement is required to transfer pile reactions to the effective sections.

Arguments similar to those for moment apply to the effective width for shear strength estimation.

C8.8.5 Concrete Piles

C8.8.5.1 Transverse Reinforcement Requirements

Note the special requirements for pile bents given in Article 8.8.2

C8.8.6 Plastic Rotation Capacities

A moment-curvature analysis based on strain compatibility and nonlinear stress-strain relations can be used to determine plastic limit states. From this a rational analysis is used to establish the rotational capacity of plastic hinges.

C8.8.6.1 Life Safety Performance

If a section has been detailed in accordance with the transverse reinforcement requirement of these provisions, then the section is said to be ‘capacity protected’ against undesirable modes of failure such as shear, buckling of longitudinal bars, and concrete crushing due to lack of confinement. The one remaining failure mode is low cycle fatigue of the longitudinal reinforcement. The fatigue life depends on the fatigue capacity [Chang and Mander, 1994a, (NCEER 94-0006)] versus demand [Chang and Mander, 1994b (NCEER 94-0013)].

This rotational capacity ensures a dependable fatigue-life for all columns, regardless of the period-dependent cyclic demand.

C8.8.6.3 In-Ground Hinges

In-ground hinges are necessary for certain types of bridge substructures. These may include, but not restricted to:

- Pile bents
- Pile foundations with strong pier walls
- Drilled shafts
- Piled foundations with oversized columns.

It is necessary to restrict these plastic hinge rotations in order to limit the crack width and plastic strains to avoid long-term corrosion problems after an earthquake has occurred. This limit is expected to reduce plastic strains to less than 40 percent of their above-round counterpart (with $\theta_p = 0.035$ rad.) This is because the plastic hinge length of in-ground hinges is typically two pile diameters due to the reduced moment gradient in the soil.

C8.9 BEARING DESIGN REQUIREMENTS

One of the significant issues that arose during the development of these provisions was the critical importance of bearings as part of the overall bridge load path. The 1995 Kobe earthquake, and others that preceded it and have occurred since, clearly showed poor performance of some very recent bearing types and the disastrous consequences that a bearing failure can have on the overall performance of a bridge. A consensus was developed that some testing of bearings would be desirable provided a designer had the option of providing restraints or permitting the bearing to fail if an adequate surface for movement is provided. A classic example occurred in Kobe where a bearing failed and it destroyed the steel diaphragm and steel girder because the girder became jammed on the failed bearing and could not move.

There has been a number of studies performed when girders slide either on specially designed bearings or concrete surfaces. A good summary of the range of the results that can be anticipated from these types of analyses can be found in Dicleli, M., Bruneau, M. (1995).

C8.9.1 Prototype and Quality Control Tests

The types of tests that are required are similar but significantly less extensive than those required for seismically isolated bridges. Each manufacturer is required to conduct a prototype qualification test to qualify a particular bearing type and size for it's design forces or displacements. This series of tests only needs to be performed once to qualify the bearing type and size, whereas on an isolated project, prototype tests are required on every project. The quality control tests required on 1 out of every 10 bearings is the same as that required for every isolator on seismic isolation bridge projects. The cost of the much more extensive prototype and quality control testing of isolation bearings is approximately 10 to 15% of the total bearing cost, which is of the order of 2% of the total bridge cost. The testing proposed herein is much less stringent than that required for isolation bearings and is expected to be less than 0.1% of the total bridge cost. However, the benefits of testing are considered to be significant since owners would have a much higher degree of confidence that each new bearing will perform as designed during an earthquake. The testing capability exists to do these tests on full size bearings. Caltrans has invested in a full size test machine located at the University of California, San Diego, and similar capabilities exist at other universities, government laboratories, and commercial facilities.

C8.10 SEISMIC ISOLATION DESIGN REQUIREMENTS

The commentary on this subject is given in C15 which will become a new section in the AASHTO LRFD specifications.

C8.11 SUPERSTRUCTURE DESIGN REQUIREMENTS

Capacity-protection or elastic design of the superstructure is required to reduce the possibility of earthquake induced damage in the superstructure. It is generally felt that such damage is not easily repairable and may jeopardize the vertical load-carrying capability of the superstructure.

The elastic forces from the 3% in 75-year/1.5 mean deterministic event may be used in lieu of capacity-protecting the superstructure, because their use will typically satisfy the performance objective for the design level ground motion.

When the superstructure can effectively span transversely between abutments as a diaphragm, then the resistance of the intermediate piers may not contribute significantly to the lateral resistance. In such cases, the elastic forces for the design earthquake should be used for the design of the superstructure lateral capacity. However, when designed in this manner, the superstructure could be vulnerable in earthquakes that produce shaking at the site that is larger than the design ground motion. If the maximum resistances of the abutments are defined, then they may be used to define the maximum forces in the superstructure, as an alternate to the use of the elastic seismic forces.

C8.11.2 Load Paths

The path of resistance for the seismic loads should be clearly defined, and the mechanisms for resistance engineered to accommodate the expected forces. In general, the seismic forces in the superstructure should be those corresponding to a plastic mechanism (yielding elements at their respective overstrength conditions) or the elastic demand analysis forces. The load path in the superstructure should be designed to accommodate these forces elastically.

Where non-seismic constraints preclude the use of certain connection elements, alternate positive connections should be made. For instance, non-composite action is often used in the negative moment regions of continuous steel plate girders. Consequently, studs are not present to transfer inertial loads from the deck to the diaphragm. In such cases, the girder pad portion of the deck slab could be extended beside the girder flange to provide a bearing surface.

Longitudinal forces may only be transferred to the abutment by a continuous superstructure. If a series of simple spans are used the seismic loads must be resisted at each substructure location.

C8.11.3 Effective Superstructure Width

In the case of longitudinal seismic force resistance, the piers will receive loads at the connection points between the superstructure and substructure. For longitudinal loading the primary load path from the superstructure to the pier is along the girder or web lines. To effectively transfer these forces to the substructure, connections to the piers should be made close to the girder or web lines. This requires that the cap beam of the pier in a single- or multi-column bent should be capable of resisting the effects of these forces, including shears, moments, and torsion.

In the case of longitudinal moment (moment about the superstructure transverse axis) transferred between super- and substructure, significant torsion may develop in the cap beam of the pier. The designer may chose to resist the longitudinal moment directly at the column locations and avoid these torsions. However, in a zone adjacent to the column, the longitudinal moment in the superstructure must then be transferred over an effective superstructure width, which accounts for the concentration of forces at the column location. The provisions used to specify the effective width are based on Caltrans' *Seismic Design Criteria* (1999). On the other hand, if the cap beam is designed for the longitudinal moments applied at the girder lines, no effective width reduction of the superstructure is required.

C8.11.4 Superstructure-To-Substructure Connections

In general the connections between the superstructure and substructure should be designed for the maximum forces that could be developed. In the spirit of capacity design, this implies that the forces corresponding to the full plastic mechanism (with yielding elements at their overstrength condition) should be used to design the connections. In cases where the full mechanism might not develop during the 3% in 75-year earthquake, it is still good practice to design the connections to resist the higher forces corresponding to the full plastic mechanism. It is also good practice to design for the best estimate of forces that might develop in cases such as pile bents with battered piles. In such bents the connections should be stronger than the expected forces, and these forces may be quite large and may have large axial components. In such cases, the plastic mechanism may be governed by the pile geotechnical strengths, rather than the piles' structural strengths.

Elements that fuse to capacity protect attached elements should be treated similarly to elements that form a plastic hinge. The overstrength force from the fusing element may be used to design the adjacent elements and connections. Just as with plastic hinging, the designer should attempt to control the failure mechanism, as much as is possible. This implies that some modes of failure may be suppressed by adding strength, and others promoted by reducing strength. In general, the upper bound strength of the fuse should be about 75 percent of capacity of the elements being protected. For instance, strength of a fusible shear key at a pile-supported abutment might be sized to be 75 percent of the lateral strength of the pile group. The connections of adjacent elements to the abutment would then be designed to provide at least this capacity.